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No. 11
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# JOURNAL
## OF THE
# AMERICAN CONCRETE INSTITUTE

Papers and Reports .....1165-1224

55-69 Qualification Plan for Ready-Mixed Concrete Plants.....C. E. PROUDLEY 1165

55-70 Fatigue Study of Air-Entrained Concrete..... JOHN DE C. ANTRIM and JOHN F. McLAUGHLIN 1173

55-71 Cylindrical Shell Analysis Simplified by Beam Method.....JAMES CHINN 1183

55-72 Flexural and Compressive Strength of Concrete as Affected by the Properties of Coarse Aggregates.....M. F. KAPLAN 1193

55-73 Internal Shielding Construction at Shippingport Nuclear Power Plant..L. EARL TABLER, JR. 1209

55-74 Further Tests of Dynamically Loaded Beams.. F. T. MAVIS and J. J. STEWART 1215

Concrete Briefs .....1225-1228

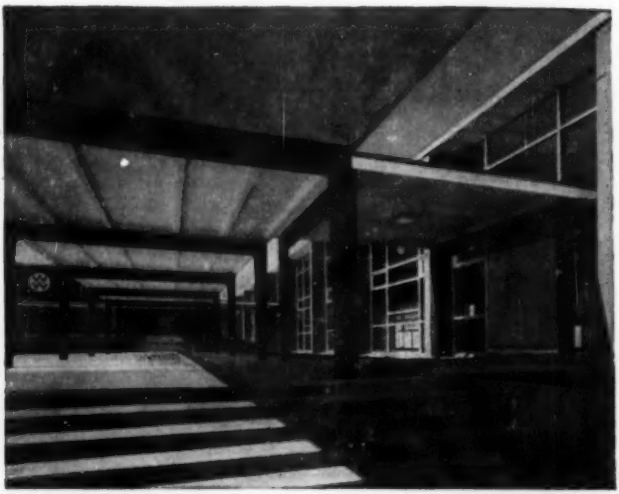
Current Reviews .....1229-1236

News Letter..... 1-40

ACI
PROCEEDINGS
Vol. 55

# TACOMA

# *Rings the Bell!*



● "As close to perfection as one could desire for student evacuation and for reducing the spread of fire." That's how the head of Tacoma's Fire Prevention Bureau described this new, all-concrete Woodrow Wilson High School.

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MAY 1959

No. 11

Vol. 30

PROCEEDINGS VOL. 55

# JOURNAL
## OF THE
# AMERICAN CONCRETE INSTITUTE

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GEORG WÄSTLUND discusses the "Use of High-Strength Steel in Reinforced Concrete" comparing Swedish high-strength Kam steel and Austrian Tor steel along with other types of high-strength steels developed in Europe.

●

Coauthors R. SHALON and M. RAPHAEL present "Influence of Sea Water on Corrosion of Reinforcement." Data are offered on over 400 reinforced mortar prisms where sea water is used in the mix.

●

"Waterstops for Joints in Concrete" by B. KELLAM and M. T. LOUGHBOROUGH discusses physical properties of waterstops of various shapes and materials; includes recent methods developed for cutting, splicing, and installation.

●

"Fatigue and Static Strength of Stud Shear Connectors" by BRUNO THÜRLIMANN presents results of push-out tests to determine fatigue behavior of stud connectors. Based on these and other tests, criteria are suggested for design of composition beams.

●

K. M. ALEXANDER and J. WARDLAW report on the "Effect of Powdered Minerals and Fine Aggregate on the Drying Shrinkage of Portland Cement Paste," studied under a wide range of conditions. An equation is presented which relates shrinkage and strength changes to changes in water and admixture contents.

●

Discussion of papers published in the October, November, and December, 1958, issues will appear in the June JOURNAL in accordance with the Institute's quarterly publication schedule for *Proceedings* discussion.

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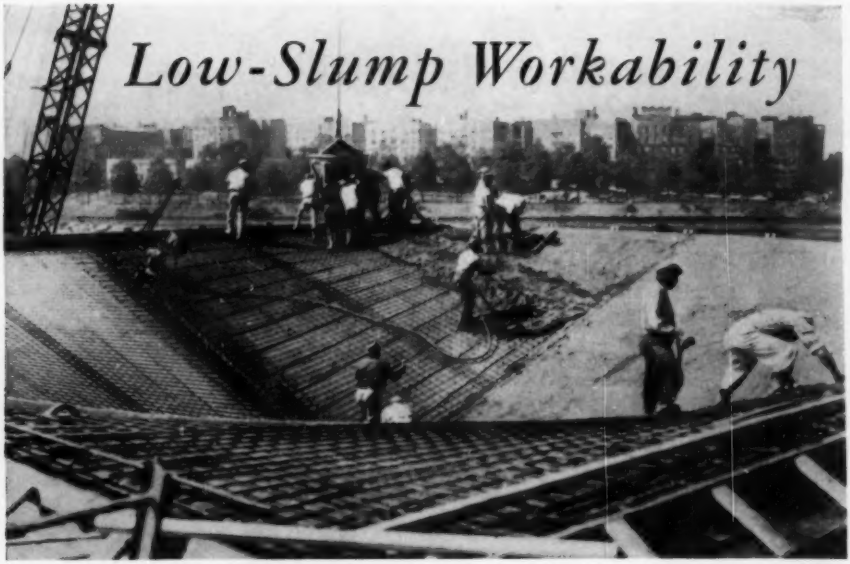


*This hyperbolic paraboloid, spanning 60 feet, is one of six for the new library of Hunter College, New York City.*

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Title No. 55-69

# Qualification Plan for READY-MIXED CONCRETE PLANTS

By C. E. PROUDLEY

**The North Carolina State Highway Commission lists ready-mixed concrete plants approved as sources of concrete for state highway work on the basis of periodic checks by its engineers and inspectors. Ratings of plants on the approved list are based on plant equipment and layout and on qualifications of the plant employees. An inspection classification further indicates whether there is need for close scrutiny by the consumer, or if the plant may be trusted to apply conscientious control of quality and uniformity. Materials used by the approved plants are inspected and tested by highway commission facilities, and a training program qualifies plant employees as concrete technicians.**

■ MORE THAN 90 PERCENT of the portland cement concrete used in structures on North Carolina state highway projects is supplied by ready-mixed concrete plants. There are currently about 107 plants operating in the state, of which 97 have been approved as sources of concrete for highway work. Several approved plants are in neighboring states close to the border.

Approval of facilities, methods and materials for concrete is the responsibility of the department of materials and tests for all highway work. Repeated use of the same ready-mixed concrete plants for state jobs made it apparent that certain plants were producing uniformly acceptable concrete while others could supply concrete meeting the highway specifications only after major rearrangement of stockpiles and equipment to comply with the details of the standard specifications. For a brief period the list of approved plants existed only in the minds of the concrete engineers and inspectors from the central laboratory.

As requests from resident engineers became more numerous for inspection of plants from which contractors informed them they intended to purchase concrete, we began listing plants whose concrete had been used successfully and that were equipped as required. These lists were distributed to the highway division offices to expedite answers to such inquiries. We recognized also a considerable number of plant owners who did not comply with specifications but this information was not listed.

**Since ready-mixed concrete today comprises a sizeable portion of the nation's concrete operations, its inspection and control assume at least proportionate importance in the area of concrete inspection. For this reason, and because ready-mixed concrete production often presents special problems in its inspection, ACI Committee 611, Inspection, is particularly pleased to endorse the approach to this type of inspection described by Mr. Proudley.**

**ACI member C. E. PROUDLEY has been state materials engineer, North Carolina State Highway Commission, Raleigh, since 1936. Mr. Proudley was head of the first portland cement research laboratory of the U. S. Bureau of Public Roads in 1925-27, and served as research engineer for the National Sand and Gravel Association and the National Ready Mixed Concrete Association 1927-36. He received his BS and CE degrees from George Washington University.**

When knowledge of the list of approved plants became more widespread many operators requested approval of their plants. Inspection of plants in the early days revealed a woeful lack of technical ability and planning in most cases. Only a few were acquainted with any type of specifications, and not a few expressed their indifference to learning about them or interest in revising their plants or equipment to supply state highway jobs. Word was not long in getting around among contractors, engineers, and architects and the ready-mixed concrete plant that was not on the state highway commission's approved list was viewed with suspicion even though the price was "right." More applications for approval began rolling in from plants which still disclaimed any interest in highway work.

## BASIS FOR APPROVAL

The procedure is to have one of the inspectors or engineers from the central laboratory visit the plant when requested and discuss the highway commission's standard specifications with the owner or manager. This first contact is usually revealing in such matters as technical ability and probable cooperation. A tour of his plant tells the rest of the story. "Housekeeping" is an important consideration in judging an operation.

The highway commission's standard specifications are the basis for plant inspection; however, a number of details are not covered by them. The ASTM C 94 specifications are referred to as supplementary to the state's standards although neither is fully comprehensive. Some requirements are imposed arbitrarily in the interest of control especially since qualified supervision and inspection during construction are likely to be absent. In other words, we make rules as we see the need for them but apply them to everyone alike and explain our purposes in establishing the requirements. We violate certain standard requirements with the same thoughtful care and reasoning.

In August 1958, Report No. 18, "Approved Ready-Mixed and Central-Mixed Concrete Plants," was released in a memorandum to state highway engineers, resident engineers, ready-mixed concrete producers, and contractors. The text of the memorandum is pertinent to a description of the methods of approving the plants, therefore, it is quoted here with but little condensation.

"The attached list shows ready-mixed concrete plants which have been inspected, and at the time of inspection were approved as a source of supply of portland cement concrete for use in state highway work.

"The engineer in charge of the work on which concrete from any of these sources is to be used should, nevertheless, examine the equipment for compliance with the requirements to be found in Section 160-3.1 on p. 188 and Section 160-3.8 on p. 201 of the 1952 'Standard Specifications for Roads and Structures.' Attention is called especially to the requirement that each transit mixer shall be equipped with a revolution counter in satisfactory working condition.

"Ready-mixed concrete is acceptable only so long as there is a representative of the commission present at the plant during batching for state work, except as otherwise provided for below, and it is his duty to verify the accuracy of the scales by means of the standard 50-lb test weights which the ready-mixed concrete operator must have available.

"Sampling and testing of cement at all approved ready-mixed concrete plants must be done in the following manner. An approved representative of the ready-mixed concrete company shall secure a ½-gal. sample of cement from each car. The sample shall be taken *from the car* and not from the bins; and shall be taken just prior to loading the cement into the bins. The sample shall be put into a friction top metal can, properly sealed against moisture, and identified as to date sampled, brand of cement and mill, car number, number of barrels of cement in car, and date cement arrived at the ready-mixed concrete plant. Each sample, shall be given a sample number, beginning with number one, and the numbers shall run consecutively for all samples taken during one year with the last two figures of the year being part of the sample number. . . .

"These samples shall be stored in a suitable place at the ready-mixed concrete plant until the total number of samples stored represents a quantity equal to twice the capacity of the bins. Then, when an additional sample is taken, the oldest of the stored samples may be discarded so long as there is maintained at all times a sufficient number of the newest consecutive samples to represent twice the capacity of the bins. . . .

"When concrete is to be delivered to state highway work, the *state highway inspector* shall, on the first day that concrete is delivered, secure from the ready-mixed concrete operator the samples representing the last two cars of cement loaded into the bin and send them to the laboratory properly identified as to project, etc., for testing. . . .

"The laboratory has inspectors who are assigned to inspection of all aggregates produced for state highway use and the fine and coarse aggregates shipped to the ready-mixed concrete producers listed, but if there is any question as to compliance of the aggregates with the applicable requirements for grading or cleanness, representative samples of the materials should be submitted to be tested before use. It is recommended that you communicate with the division materials inspector for the division in which the work is located.

"Ready-mixed concrete producers listed are requested to always insist on being furnished state highway inspected aggregates whenever placing their orders.

"The correct proportions for various combinations of aggregates should be secured from the Raleigh laboratory.

"Consideration for the problems of the ready-mixed concrete operator is urged, especially in the matter of notifying him of anticipated concrete requirements *at least 24 hr in advance.* Preliminary examination of equipment and materials should be made sufficiently in advance to permit him to rectify any irregularities before concrete is needed. Cooperation between the engineer, the contractor, and the ready-mixed concrete producer should result in a high degree of efficiency and good quality of concrete. The laboratory has representatives who will be glad to assist at all times.

"The method of classification of ready-mixed concrete plants classifies a plant 'A' or 'B' on the basis of the plant equipment, and '1' or '2' on the basis of the inspection and control that is to be applied to the plant.

# CHECK LIST FOR INSPECTION OF TRANSIT AND CENTRAL MIX CONCRETE PLANTS

Department of Materials and Tests
North Carolina State Highway Commission
Raleigh, North Carolina

PLANT INSPECTED: _____ DATE: _____

LOCATION: _____

OFFICIALS: _____

I Type plant: Transit_____ Central_____

II Stockpile:
Condition: Segregated Yes_____ No_____ Foreign matter: Yes_____ No_____
Method of handling aggregate: Belt_____ Crane_____ Bucket_____ Properly partitioned: Yes_____ No_____
Sand stockpiled 12 hours or more. Yes_____ No_____
Stone stockpiled 12 hours or more. Yes_____ No_____
Sand stockpiled at: RM plant_____ Pit_____
Stone stockpiled at: RM plant_____ Pit_____ Quarry_____
Does RM plant stockpile nonspecification materials? Yes_____ No_____
If yes, are these materials properly separated from specification materials? Yes_____ No_____

III Aggregates:
Source of supply of fine aggregate: _____ Source of supply of coarse aggregate: _____
Are all materials ordered to comply with North Carolina State Highway Specifications? Yes_____ No_____

IV Scales
Cement:
Bag_____ Bulk_____
Dial_____ Capacity_____ Increment_____
Beam_____ Capacity_____ Increment_____ Number of beams_____
Do scales comply with specifications? Yes_____ No_____
Aggregates:
Dial _____ Capacity_____ Increment_____
Beam_____ Capacity_____ Increment_____ Number of beams_____
Do scales comply with specifications? Yes_____ No_____
Separate scales for cement and aggregate? Yes_____ No_____
One stop_____ Two stop_____ (for truck-mixer loading)
Over-all condition of scales (knife-edges, etc.) _____

V Bins and hoppers:
Number of bin compartments for sand_____ Number of bin compartments for stone_____
Number of bin compartments for gravel_____ Number of bin compartments for others_____ What others?_____
Maximum size batch_____
Cement hopper equipped with vibrator. Yes_____ No_____ Aggregate hopper equipped with vibrator. Yes_____ No_____
Operation of hoppers is satisfactory: Yes_____ No_____ If "No," why _____
Type of cement bin_____, number of bins_____, capacity of each bin_____

VI Water:
Type of water measuring device:
Scales_____ Meter_____ Truck mixer tanks_____ Gravity tank_____
Automatic_____ Manual_____ Meets specifications Yes_____ No_____

VII Admixtures:

| Brands used for: | Automatic | Manual |
|---|---|---|
| (a) air entrainment | | |
| (b) retarder | | |
| (c) accelerator | | |

VIII Mixers:
Central mixer:
Condition of drum_____ Condition of blades_____
Does automatic timing device meet specifications? Yes_____ No_____
Truck mixers:
Number of mixers equipped with counters_____ Type of counters used_____
Mixers added to fleet since last inspection_____
Mixers deleted from fleet since last inspection_____
Mixers now failing to meet specifications:

| Truck No. | Reasons for failure |
|---|---|
| _____ | _____ |
| _____ | _____ |

IX Testing of materials:
Cement tested by_____ Samples taken at_____
Concrete proportions for non-state work designed by_____
Tests of concrete made by_____ Records available of tests covering period from_____
Tests of aggregates performed at plant by_____

X Testing equipment complying with specifications:
Test weights. Yes_____ No_____
(See pp. 4 and 5, paragraphs (a) through (p), of the *Proposed Basis for Classification of Approved Ready-Mixed Concrete Plants*)

XI General attitude of personnel_____
General condition of plant_____

XII Any further remarks_____

"An 'A' classification for equipment indicates that all equipment complies with all requirements of the standard specifications. (See check list, p. 1168.) A 'B' classification indicates that one or more pieces of equipment vary from specification requirements, but the use of this equipment has been approved provided it is used in a manner designated by the department of materials and tests.

"An inspection classification of '1' indicates that the plant employs a full-time concrete technician who has been certified by the state highway commission and whose sole duty, while concrete is being delivered to state highway work, is to have charge of and exercise close supervision over the production and control of the concrete.

"Plants which do not employ a certified concrete technician are given an inspection rating of '2,' and must have a state highway inspector present when batching out for state highway work.

"Any plant may improve its rating at any time by complying with the appropriate requirements.

"The two columns [in the list of approved plants] headed 'Approved Transit Mixers' indicate the number of approved mixer units, and the capacity of each, available at each ready-mixed concrete plant. This information is included primarily for use of engineers and contractors purchasing concrete for use in state highway work in estimating the rate of delivery of concrete that may be expected from any plant.

"Ready-mixed concrete plant managers should notify the department of materials and tests of any changes to be made in the information furnished in the list. Revised lists will be distributed from time to time."

## HOW THE REGULATIONS ARE APPLIED

A brief discussion of some of the paragraphs in the above quoted memorandum accompanying the list of plants may be of interest. Inspectors and engineers under the supervision of the Raleigh laboratory make the rounds of the approved plants and with the aid of the check list shown on the opposite page reinspect for continued compliance with requirements for approval. It is not unusual to put a plant on notice that it will be removed from the list unless corrections of inadequate conditions are made immediately. Sometimes the local resident engineer detects some irregularity and notifies the laboratory or its representative.

### Inspection of materials

The method of testing cement is recognized as considerably less than 100 percent effective but the spot check method has served to keep the cement mills alert to the necessity of shipping only that product which will pass all tests since they never know what car will be tested by the state highway laboratory, and they are extremely jealous of their reputations for consistent quality. We are sure to test some of the cement being used in each project and considering the volume of cement used through ready-mixed concrete plants it is probable that we test some from every bin used to supply North Carolina.

Highway materials inspectors maintain close control and records of the aggregates produced throughout the state and nearby plants in other states. By examination of shipping schedules at the aggregate plants, notification is given to the local inspectors at the various ready-mixed concrete plants in other highway divisions of the approval of the aggregates for grading, clean-

ness, and quality. Although it may appear that much nonhighway inspection is performed, it does not involve an appreciable increase in the inspector's obligations or work. We admit performing valuable service to the concrete producers at no expense to them but it appears to be the most efficient method under prevailing circumstances.

**Delivery schedules**

Scheduling of delivery of concrete is recognized as one of the most important factors in securing a satisfactory job. We recommend but can do little to enforce the giving of ample advance notice since this is by agreement between the contractor and his supplier. However, if it is obvious that the ready-mixed concrete plant is the cause of continual delays, that plant is removed from the approved list.

**Plant facilities**

Classification of plants to give some indication of their ability to supply concrete of uniform characteristics meeting specification requirements must give separate consideration to plant facilities and personnel since either one alone cannot guarantee good concrete. Plants that do not have reliable scales, water measuring devices, uncontaminated stockpiles, and other fundamental facilities are not approved. If, however, the scales are accurate but the same dial scale is used to weigh both cement and aggregates, the plant may be approved if no other violation of the standard requirements exists. Also, if there are three or more transit mixers fully approved the plant may have any number of truck mixers that are not approved so long as no unapproved mixer is used for state highway work. Several other nonstandard conditions are allowed but in every instance the plant management is urged to change to standard equipment as soon as economically feasible and, generally, they do.

Stockpiling aggregates and handling cement are items of major importance and some of the small plants are watched with more than average inspection for careless handling. Several warnings may be followed by disapproval even though the plant has no contract for state highway work.

**Personnel training**

A campaign has been in progress for over 6 years to train ready-mixed concrete plant personnel in all phases of their responsibilities with considerable success. Several 3-day sessions are held each winter in the Raleigh laboratory. Plant operators and state highway employees are urged to attend. Instruction is basic and those who wish may take an examination for registration by the highway commission as "Certified Concrete Technician." The examination requires about 3 days and consists of two written parts, one with and the other without text books, on matters of proportioning and adjustment of concrete mixtures; another part to demonstrate ability to perform tests of aggregates and concrete properly; and a final demonstration of how to apply quality control to a ready-mixed concrete operation.

Although the number of individuals certified is not as great as is desired the interest in plant control has been encouraging. The attitude of the industry has been very cooperative, and the reduction in complaints from contractors and highway engineers is reassuring. Much credit must be given to the concrete engineers and inspectors from the department of materials and tests as well as to the progressive management of the plants among whom there is generally maintained a high degree of mutual respect.

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**Discussion of this paper should reach ACI headquarters in triplicate by Aug. 1, 1959, for publication in the Part 2, December 1959 JOURNAL.**

Title No. 55-70

# Fatigue Study of Air-Entrained Concrete

By
JOHN DE C. ANTRIM
JOHN F. MCLAUGHLIN

**Fatigue tests were performed on two types of concrete, each proportioned for the same 28-day compressive strength; one contained only "accidental" air, while the other contained intentionally entrained air which was maintained at a constant level. There was little variation in the ages of the specimens which were tested at stress levels of 50, 60, 70, 80, and 90 percent of the ultimate static compressive strength of the respective mixes. Within the limits of the investigation, the fatigue behavior of air-entrained plain concrete is similar to that of non-air-entrained plain concrete. However, the air-entrained concrete was more uniform than the non-air-entrained concrete with regard to both fatigue and static strength properties.**

■ THERE HAS BEEN AN INCREASED NEED for information concerning the behavior of concrete under repeated applications of loads that are less than the ultimate. This type of loading, known as a fatigue loading, does not occur as frequently as the steady or static type, but it is just as important a factor in design for those cases where it is present.

Experience has shown that it is insufficient for the designer to know the fatigue strength of the concrete at a specified fatigue life, or in the case of certain metals, the value of the fatigue limit; he needs to know the shape of the *S-N* curve ($S$ = testing stress level; $N$ = number of cycles for failure). From the *S-N* curve he can relate his particular working stresses to a fatigue life or, knowing the number of cycles of stress a structure will receive in its lifetime, he can select the maximum stress which will not cause failure. The particular fatigue limit of concrete which is of greatest concern is that limit for cycles of stress varying from zero to a maximum in one direction only.

Although there has been no information reported on the fatigue behavior of air-entrained concrete, it is known from many studies[1] that many properties of concrete are altered by the addition of entrained air; hence, the question naturally arises as to what effect entrained air has on the fatigue properties of concrete. Work reported in this paper had as its objective the establishment and comparison of *S-N* curves for non-air-entrained concrete and air-entrained concrete.

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JOHN F. MCLAUGHLIN, is assistant professor of civil engineering at Purdue University, where he has been a staff member continuously since 1950. An ACI member, Professor McLaughlin teaches courses in transportation and highway engineering, and has been active in research on portland cement concrete, mineral aggregates, and bituminous materials. He serves on technical committees for ACI, ASTM, and the Highway Research Board.

Most of the investigations concerned with the fatigue behavior of plain concrete were conducted during 1900 to 1928. Not many general conclusions can be drawn regarding the fatigue of concrete since an analysis of the results of these early investigations and also the more recent ones, shows that the methods of investigating the problem varied considerably. Loading arrangement, rate of loading, type and size of specimen, and age of specimen are only a few of the differences existing between investigations. Nordby[2] in his review of research on fatigue of concrete has summarized the results of these investigations.

## TESTING PROGRAM

The tests required for this study were done in two parts. The first part, tests on specimens made from a non-air-entrained concrete mix, will be referred to as the FN series. The second part, tests on specimens made from an air-entrained concrete mix, will be referred to as the FA series. Each series consisted of eight batches made periodically during the course of the investigation. In general, a batch yielded about 25 concrete cylinders, 3 in. in diameter by 6 in. high.

### Materials, proportioning

The coarse aggregate was a crushed limestone from central Indiana, maximum size ½-in. The fine aggregate was a local sand obtained from a river terrace deposit. Type I portland cement from a single clinker batch was used in both mixes and Darex, added at the mixer, was used as the air-entraining agent for the air-entrained concrete.

The non-air-entrained mix was proportioned for a strength of 4000 psi and a slump of 3 in. The mix had a cement factor of 5.4 sacks per cu yd of concrete and a water-cement ratio of 0.69 by weight.

The air-entrained mix was proportioned for a strength of 4000 psi, a slump of 3 in., and an air content of 7 percent. The mix had a cement factor of 5.0 sacks per cu yd of concrete and a water-cement ratio of 0.61 by weight.

After 28 days of curing which followed the procedure specified in ASTM C 129-52T, the specimens underwent 3 to 4 days of oven drying at approximately 220 F to dry the concrete and prevent further hydration. Soon after removal from the oven, each specimen was capped on both ends with a sulfur compound and then stored under room conditions until testing.

### Test methods

Shortly after capping, eight specimens from each batch were tested for their ultimate static compressive strength. The average ultimate strength of these eight specimens served as an estimate of the ultimate strength of the batch, and the stress levels at which fatigue tests were performed were based on this estimate of batch strength. Following completion of the fatigue tests on each batch, the remaining specimens in the batch were tested for their ultimate static compressive strength. The object of these later compression tests was to determine if there was any further strength gain in the concrete during the time taken by the fatigue tests.

The fatigue tests were conducted, when conditions permitted, at stress levels of 50, 60, 70, 80, and 90 percent of the estimated ultimate strength of the batch. A minimum stress of approximately 70 psi was maintained regardless of the magnitude of the maximum stress.

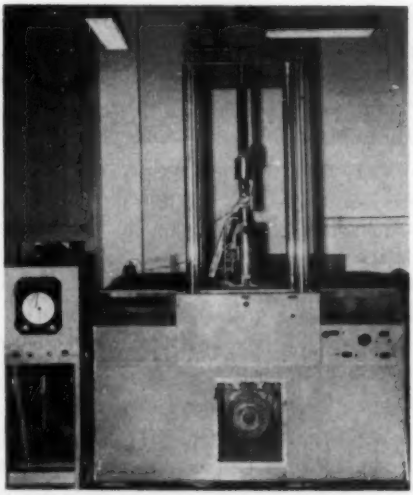

Fig. 1—Krouse-Purdue axial-load fatigue machine

The process of establishing the desired maximum and minimum loads on the fatigue testing machine required a considerable number of cycles; therefore, the loading was set by first inserting a dummy specimen in the machine. When the loads were established, the machine was stopped and the dummy specimen was replaced by a test specimen. On restarting the machine, usually little or no adjustment was necessary to obtain the desired loads. In every case, a virgin specimen was used to obtain the information that is reported in this study. If a test was interrupted for any reason, it was started over again with a new specimen.

The Krouse-Purdue axial-load fatigue machine used in this study (Fig. 1) is of the constant deflection type and derives its force from hydraulic pressure acting on a large piston directly connected to the test piece through a piston rod. The load applied by the machine consists of two components: a preload and a pulsating load. The preload is directly proportional to the average differential pressure existing between the two ends of the hydraulic cylinder, and it is controlled by automatically controlling make-up oil pressures. The amount by which the pulsating load varies above and below the preload depends on a variable-throw crank that can be adjusted before or while the machine is operating. The operating speed of the fatigue machine is 1000 cycles per min and the machine has a maximum loading capacity of ± 60,000 lb. Magnitude of the applied loads is measured by means of an electronic system actuated by a load cell that is a part of the lower end of the load screw.

## DISCUSSION OF RESULTS

The results of this study and the discussions of these results have been divided into two parts. The first part is concerned with the analyses of the physical properties of the plastic concrete and the hardened concrete used in this research work. The second part covers the results of the fatigue testing program.

### Analysis of mix data

The purpose of the study was to compare the fatigue behavior of non-air-entrained and air-entrained concretes that were similar except for air con-

tent. Since the concretes were prepared in batches, the properties of the two mixes had to be estimated from the corresponding batch properties. These batch properties, slump and air content of the plastic concrete, and strength of the hardened concrete at various ages expressed as means, are shown in Tables 1 and 2.

The statistical procedure known as the analysis of variance was used for testing for significant differences among the means. Except for some minor adjustments in the tests because of the nature of the data being tested, the data summarized in Tables 1 and 2 were tested by the same methods. The non-air-entrained mix and the air-entrained mix were treated separately and then a comparison was made of the two mix strengths. All statistical tests were made at the 5 percent significance level. The results of these statistical tests on data collected from over 200 specimens are best summarized as follows:

1. The mean strengths of the non-air-entrained concrete mix and the air-entrained mix, 4090 and 4430 psi, respectively, were not significantly different. This statement is based on a $t$-test at an alpha level of 5 percent.

**TABLE 1—PHYSICAL CHARACTERISTICS OF MIX FN (NON-AIR-ENTRAINED)**

| Batch designation | Air content, percent | Slump, in. | Average ultimate compressive strength at 7 days (3 specimens tested), psi | Average ultimate compressive strength after oven drying* (8 specimens tested), psi | Average ultimate compressive strength after completion of fatigue tests† (7 to 8 specimens tested), psi |
|---|---|---|---|---|---|
| FN1 | 0.6 | 3 | None tested‡ | 4070 | 3970 |
| FN2 | 1.2 | 3½ | None tested‡ | 3560 | 3520 |
| FN3 | 1.8 | 6 | 2690§ | 3440 | None tested** |
| FN4 | 1.5 | 2½ | 2510 | 4010 | 4150 |
| FN5 | 0.5 | 3 | 2470 | 4150 | 4120 |
| FN6 | 0.1 | 2½ | 2980 | 4270 | 4080 |
| FN7 | 0.8 | 6 | 2760 | 3820 | 3980 |
| FN8 | 1.0 | 6 | 2780 | 4230 | 4180 |

*Specimen age ranged from 33 to 36 days.
†Specimen age ranged from 47 to 65 days.
‡Specimens used in the fatigue tests.
§Specimens tested at 11 days.
**Specimens tested at a later date.

**TABLE 2—PHYSICAL CHARACTERISTICS OF MIX FA (AIR-ENTRAINED)**

| Batch designation | Air content, percent | Slump, in. | Average ultimate compressive strength at 7 days (3 specimens tested), psi | Average ultimate compressive strength after oven drying* (8 specimens tested), psi | Average ultimate compressive strength after completion of fatigue tests† (7 to 8 specimens tested), psi |
|---|---|---|---|---|---|
| FA1 | 7.6 | 1¾ | 2560 | 4020 | None tested‡ |
| FA2 | 10.5 | 2¾ | 2740 | 4380 | 4260 |
| FA3 | 7.7 | 2 | 2860 | 4490 | 4410 |
| FA4 | 7.5 | 1¾ | 2900 | 4540 | 4430 |
| FA5 | 7.7 | 2 | 2710 | 4300 | 4250 |
| FA6 | 7.6 | 1¾ | None tested§ | 4620 | 4560 |
| FA7 | 9.0 | 2 | 2670 | 4500 | 4160 |
| FA8 | 8.2 | 2¼ | 2490** | 4170 | 3910 |
| FA9 | 8.5 | 2½ | 2640 | 4620 | 4510 |

*Specimen age ranged from 34 to 36 days.
†Specimen age ranged from 43 to 63 days.
‡Fatigue tests not run, machine out of operation for a month.
§Scheduling difficulties.
**Specimens tested at 6 days.

2. The average air content of the non-air-entrained mix was 0.9 percent and the average air content of the air-entrained mix was 8.3 percent. The coefficient of variation of the air content for the non-air-entrained mix was an abnormally high 58 percent as against a coefficient of 12 percent for the air-entrained mix.

3. Average slumps of the non-air-entrained and the air-entrained mixes were 4 and 2 in., respectively, with corresponding coefficients of variation of 40 and 15 percent.

4. There was no significant gain in strength of the non-air-entrained concrete during the periods of fatigue testing.

5. There was a significant decrease in the strength of the air-entrained concrete during the periods of fatigue testing. The decrease averaged 140 psi; however, it was assumed that this change did not materially affect the results of the fatigue tests.

## Analysis of the fatigue data

Analysis of the fatigue data given in Tables 3 and 4 was limited by the scatter that is characteristic of fatigue test results. The actual range of stress cycles endured is unknown for both the 50 and 60 percent stress levels of both the non-air-entrained concrete and the air-entrained concrete because the testing was stopped when the specimens had sustained ten million cycles without failing. There is, of course, a lower boundary for the data representing

TABLE 3—FATIGUE TEST RESULTS, NON-AIR-ENTRAINED CONCRETE

| Batch designation | Specimen number | Maximum fatigue load, lb | Minimum fatigue load, lb | Number of stress cycles endured |
|---|---|---|---|---|
| FN1 | 1 | 14,500 (52)* | 500 (1.8)* | 10,155,000→† |
| | 2 | 17,000 (60) | 500 (1.8) | 10,355,000→ |
| | 3 | 19,500 (69) | 700 (2.5) | 12,000 |
| | 4 | 22,700 (81) | 700 (2.5) | 282,000 |
| | 5 | 25,400 (90) | 500 (1.8) | 100 |
| FN2 | 1 | 12,300 (50) | 500 (2.0) | 10,568,000→ |
| | 2 | 14,800 (60) | 500 (2.0) | 10,106,000→ |
| | 3 | 17,800 (72) | 500 (2.0) | 31,000 |
| | 4 | 19,500 (79) | 500 (2.0) | 2,200 |
| | 5 | 22,200 (90) | 1000 (4.0) | 900 |
| FN3 | 1 | 12,300 (51) | 300 (1.2) | 10,330,100→ |
| | 2 | 14,400 (60) | 500 (2.1) | 3,630,400 |
| | 3 | 16,500 (69) | 700 (2.9) | 25,800 |
| | 4 | 18,700 (78) | 700 (2.9) | 19,800 |
| | 5 | 20,500 (85) | 500 (2.1) | 300 |
| FN4 | 1 | 14,100 (49) | 500 (1.8) | 10,562,000→ |
| | 2 | 16,500 (58) | 500 (1.8) | 10,472,000→ |
| | 3 | 19,000 (67) | 500 (1.8) | 2,349,600 |
| | 4 | 22,600 (79) | 800 (2.8) | 8,900 |
| | 5 | — | — | Not tested |
| FN5 | 1 | — | — | Not tested |
| | 2 | 17,400 (60) | 500 (1.7) | 10,740,000→ |
| | 3 | 20,300 (70) | 500 (1.7) | 99,700 |
| | 4 | 22,500 (78) | 500 (1.7) | 1,500 |
| | 5 | 26,100 (90) | 700 (2.4) | 3,700 |
| FN6 | 1 | — | — | Not tested |
| | 2 | 18,000 (62) | 500 (1.7) | 1,308,400 |
| | 3 | 21,000 (72) | 500 (1.7) | 278,700 |
| | 4 | 24,000 (82) | 800 (2.7) | 1,000 |
| | 5 | 26,500 (91) | 500 (1.7) | 300 |
| FN7 | 1 | — | — | Not tested |
| | 2 | — | — | Not tested |
| | 3 | 18,800 (69) | 500 (1.8) | 54,500 |
| | 4 | 21,500 (79) | 1000 (3.7) | 4,600 |
| | 5 | 24,200 (89) | 500 (1.8) | 10,900 |
| FN8 | 1 | — | — | Not tested |
| | 2 | 17,800 (61) | 500 (1.7) | 1,388,700 |
| | 3 | 20,500 (70) | 800 (2.7) | 52,200 |
| | 4 | 23,500 (80) | 1000 (3.4) | 1,700 |
| | 5 | 26,700 (91) | 500 (1.7) | 500 |

*Figure in parenthesis is the fatigue load expressed as a percentage of the average ultimate compressive strength of the batch. Average strength based on 15 to 16 specimens except for Batch FN3 which had 8 specimens.
†→indicates that specimen had not failed when test was stopped.

the 60 percent stress level. It was felt that any curve fitted to all the data for each type of concrete would undoubtedly be a poor approximation of the true curve; therefore, the curves that were drawn only represent the relationship that exists at the 70 and 80 percent levels.

Referring to the two *S-N* diagrams in Fig. 2 and 3, the position of the points on the right hand portion of both diagrams suggests that it is unlikely that a definite fatigue limit exists in the vicinity of 10 million cycles for either type of concrete. Fatigue test results for the air-entrained concrete are characterized by less scatter than the test results for the non-air-entrained concrete.

*Comparison of the FA mix to the FN mix*—A nonparametric statistical technique was used in comparing the fatigue test results from the two mixes, because the type of distribution to which the data belonged was unknown. In this analysis a method called "Runs" was used.[3] This technique, when used to test the data to which it was applicable, indicates that there is reliable evidence at the 5 percent significance level that at the respective stresss levels the data are from the same population.

*Linear regression analysis*—A consequence of plotting the cycles to failure to a logarithmic scale was a contraction or drawing together of the *N*-values. The data in this transformed

## TABLE 4—FATIGUE TEST RESULTS, AIR-ENTRAINED CONCRETE

| Batch designation | Specimen number | Maximum fatigue load, lb | Minimum fatigue load, lb | Number of stress cycles endured |
|---|---|---|---|---|
| FA2 | 1 | — | — | Not tested |
| | 2 | 18,350 (61)* | 500 (1.7)* | 10,048,000—→† |
| | 3 | 21,550 (71) | 550 (1.8) | 702,200 |
| | 4 | 23,900 (79) | 500 (1.7) | 2,800 |
| | 5 | Loads not verified | | ~100 |
| FA3 | 1 | 15,800 (51) | 400 (1.3) | 10,314,000—→ |
| | 2 | 19,200 (62) | 500 (1.6) | 6,294,500 |
| | 3 | 22,350 (72) | 500 (1.6) | 109,400 |
| | 4 | 25,300 (81) | 500 (1.6) | 600 |
| | 5 | 28,800 (93) | 500 (1.6) | 300 |
| FA4 | 1 | — | — | Not tested |
| | 2 | 19,000 (60) | 600 (1.9) | 8,022,600 |
| | 3 | 22,000 (70) | 450 (1.4) | 204,800 |
| | 4 | 25,300 (80) | 600 (1.9) | 3,500 |
| | 5 | Loads not verified | | ~100 |
| FA5 | 1 | — | — | Not tested |
| | 2 | 18,000 (60) | 600 (2.0) | 9,092,900 |
| | 3 | 21,300 (71) | 400 (1.3) | 1,683,800 |
| | 4 | 24,200 (81) | 500 (1.7) | 1,500 |
| | 5 | Loads not verified | | ~100 |
| FA6 | 1 | — | — | Not tested |
| | 2 | 19,500 (61) | 500 (1.6) | 10,157,000—→ |
| | 3 | 22,300 (69) | 600 (1.9) | 226,900 |
| | 4 | 25,900 (81) | 500 (1.6) | 3,900 |
| | 5 | — | — | Not tested |
| FA7 | 1 | 15,600 (51) | 600 (2.0) | 10,105,000—→ |
| | 2 | 18,800 (62) | 500 (1.6) | 10,256,000—→ |
| | 3 | 22,100 (73) | 500 (1.6) | 361,800 |
| | 4 | 24,900 (82) | 500 (1.6) | 600 |
| | 5 | — | — | Not tested |
| FA8 | 1 | 14,400 (51) | 700 (2.5) | 10,760,000—→ |
| | 2 | 17,400 (62) | 650 (2.3) | 10,109,000—→ |
| | 3 | 20,500 (72) | 500 (1.8) | 215,700 |
| | 4 | 23,600 (83) | 300 (1.1) | 900 |
| | 5 | — | — | Not tested |
| FA9 | 1 | — | — | |
| | 2 | 19,400 (61) | 500 (1.6) | 10,034,000—→ |
| | 3 | 22,700 (71) | 400 (1.3) | 534,900 |
| | 4 | 26,000 (82) | 500 (1.6) | 400 |
| | 5 | — | — | Not tested |

*Figure in parenthesis is the fatigue load expressed as a percentage of the average ultimate compressive strength of the batch. Average strength based on 15 to 16 specimens.

†—→indicates that specimen had not failed when test was stopped.

NOTE: Batch FA1 not tested. Fatigue machine under repair.

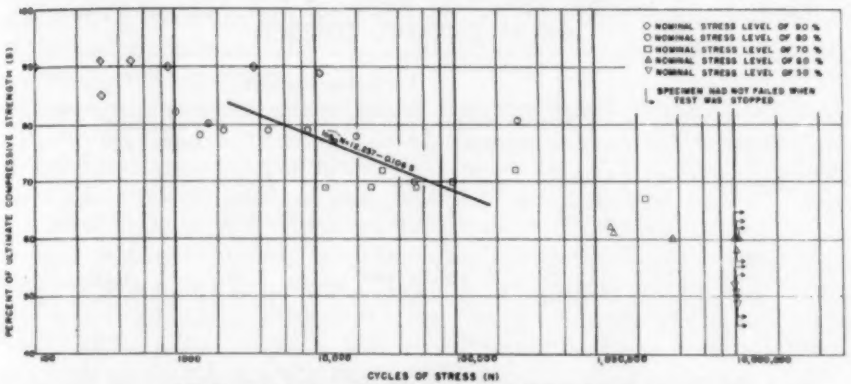
NOMINAL STRESS LEVEL OF 90 %
NOMINAL STRESS LEVEL OF 80 %
NOMINAL STRESS LEVEL OF 70 %
NOMINAL STRESS LEVEL OF 60 %
NOMINAL STRESS LEVEL OF 50 %
SPECIMEN HAD NOT FAILED WHEN TEST WAS STOPPED
PERCENT OF ULTIMATE COMPRESSIVE STRENGTH (S)
CYCLES OF STRESS (N)

Fig. 2—S-N diagram for non-air-entrained concrete

condition appear to have a normal distribution with respect to cycles to failure. It is on this assumption that the following analyses were performed.

Only the data for the 70 and 80 percent stress levels were considered in the analyses because it is only at these levels that an equal number of specimens from each mix had been tested. Fitting a curve to the data of each mix was accomplished by the method of least squares.

The least squares linear regression equation resulting from an analysis of the data for the non-air-entrained concrete is:

$$\widehat{\log_{10} N} = 12.257 - 0.106\,S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where $N$ is cycles to failure and $S$ is percent of the ultimate stress. This equation can only be used to estimate $N$ when the value of $S$ is between 66 and 84 percent since this is the range of the $S$-values used in the analysis.

The least squares linear regression equation resulting from an analysis of the data for the air-entrained concrete was calculated to be:

$$\widehat{\log_{10} N} = 20.501 - 0.214\,S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The same limits on $S$ that apply to the equation for the non-air-entrained concrete apply to this equation when estimating $N$.

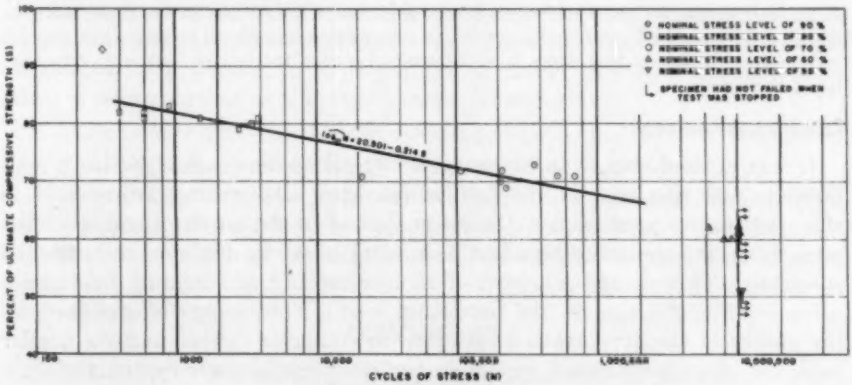
NOMINAL STRESS LEVEL OF 90 %
NOMINAL STRESS LEVEL OF 80 %
NOMINAL STRESS LEVEL OF 70 %
NOMINAL STRESS LEVEL OF 60 %
NOMINAL STRESS LEVEL OF 50 %
SPECIMEN HAD NOT FAILED WHEN TEST WAS STOPPED
PERCENT OF ULTIMATE COMPRESSIVE STRENGTH (S)
CYCLES OF STRESS (N)

Fig. 3—S-N diagram for air-entrained concrete

TABLE 5—95 PERCENT PREDICTION INTERVALS FOR NON-AIR-ENTRAINED AND AIR-ENTRAINED CONCRETE

| Stress level, percent | Prediction intervals | | | |
|---|---|---|---|---|
| | Non-air-entrained concrete | | Air-entrained concrete | |
| | Lower limit, cycles | Upper limit, cycles | Lower limit, cycles | Upper limit, cycles |
| 80 | $9.67 \times 10^{-5}$ | $3.26 \times 10^{13}$ | 8.94 | $5.17 \times 10^{7}$ |
| 75 | $1.41 \times 10^{-5}$ | $5.51 \times 10^{13}$ | 1.53 | $5.24 \times 10^{8}$ |
| 70 | $7.00 \times 10^{-4}$ | $3.14 \times 10^{14}$ | 12.18 | $9.08 \times 10^{9}$ |

Along with the linear regression equations, the correlation coefficients for the two sets of data were calculated. The degree of association among the data for the air-entrained concrete is high, the correlation coefficient being $-0.94$. The degree of association for the non-air-entrained concrete is less than that for the air-entrained data, the correlation coefficient in this case being $-0.62$. A significance test for differences between correlation coefficients performed at the 5 percent significance level indicates that there is reliable evidence that the two correlation coefficients are significantly different. This test only compares the linear association of the data and does not compare the curves determined by the regression analysis. However, it does show that the fatigue data for the air-entrained concrete have considerably greater uniformity than the fatigue data for the non-air-entrained concrete.

A test performed to determine whether the slopes of the two equations were significantly different indicates that at the 5 percent significance level there is no reliable evidence that the slopes are significantly different. This indicated equality of the slopes suggests strongly that the relationship existing between the stress level and the cycles to failure for the non-air-entrained concrete is the same as the relationship for the air-entrained concrete.

*Prediction intervals*—Although the estimating equations for both mixes have been calculated, it must be realized that if a number of tests were to be performed at a given $S$-value, the observed $N$'s would cluster about the calculated or estimated $N$. It is, therefore, advisable to determine the limits of the predicted value of an individual $N$ for a given $S$-value.

Ninety-five percent prediction intervals were calculated for each of the two $S$-$N$ curves. As shown in Table 5, these intervals are extremely wide, especially in the case of the non-air-entrained concrete. It can be seen that for a given $S$ value the predicted $N$ for the air-entrained concrete lies within the limits for the predicted $N$ for the non-air-entrained concrete. The fact that there are overlapping prediction intervals along with essentially equal slopes is good reason to assume that there is no difference between the fatigue behavior of the two types of concrete.

### Other observations

It was noticed that if an air-entrained specimen was exchanged for a non-air-entrained specimen in the fatigue machine without any adjustment in the load control mechanisms, the loads applied to the air-entrained specimen were different from those that had been established for the non-air-entrained specimen. This change consisted of an increase in the minimum load and a corresponding decrease in the maximum load. The change is explained by the ability of the air-entrained specimen to compress farther under a similar load; i.e., the air-entrained specimens probably had a lower modulus of elasticity than the non-air-entrained specimens. The desired loads were easily

obtained by increasing the crank throw, which in turn increased the travel of the loading piston and thereby compensated for the decrease in height of the air-entrained specimen.

Another phenomenon noticed was an indicated change in the applied load while a test was in progress. This was noticed only during the testing of air-entrained specimens and it was not detected for all specimens. In those cases where the load change was detected, it was only after the specimens had been subjected to 1 million or more cycles. A strange feature was that a number of the air-entrained specimens endured 10 million cycles without any load changes occurring. These changes in the loadings consisted of an increase in the minimum load and an increase or a decrease in the maximum load. The changes, however, were of different magnitudes ranging from a few hundred pounds to over a thousand pounds.

### Summary of fatigue test results

The analyses of the results of 65 specimens tested in fatigue indicate the following:

1. The portion of the $S$-$N$ curve for the non-air-entrained concrete between the stress levels of 66 and 84 percent has, as determined by a linear regression analysis, the relationship:

$$\widehat{\log_{10} N} = 12.257 - 0.106\,S$$

The portion of the $S$-$N$ curve for the air-entrained concrete between the stress levels of 66 and 84 percent has, as determined by a linear regression analysis, the relationship:

$$\widehat{\log_{10} N} = 20.501 - 0.214\,S$$

It was found, however, that the slopes of these lines are not significantly different and, in addition, that one prediction interval lies within the other. On this basis it may be said that the relationship of stress level to cycles to failure is not significantly different for the two types of concrete when a fluctuating compressive stress condition is maintained and the minimum stress is held constant at slightly in excess of zero stress.

2. The fatigue test data for the air-entrained concrete showed considerably less variation than the data for the non-air-entrained concrete.

3. The fatigue test data accumulated in this study suggest that the $S$-$N$ curve is continually sloping downward; that is, there is no indication of a definite fatigue limit for either the non-air-entrained or the air-entrained concrete.

## CONCLUSIONS

On the basis of a laboratory investigation of fatigue of concrete in which 3 x 6-in. cylinders were tested by a fluctuating compressive stress in which

the maximum was varied and the minimum stress was maintained at a constant value near zero, the following conclusions appear to be justified:

1. The fatigue behavior of non-air-entrained plain concrete and air-entrained plain concrete are not significantly different.

2. It appears that there is considerably less variation present among fatigue test data for the air-entrained plain concrete than there is for the non-air-entrained plain concrete.

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Title No. 55-71

**THE "BEAM METHOD" of analyzing cylindrical shells is briefly explained. Formulas are presented for analyzing single shells without edge beams and interior barrels of multiple shells. A numerical example illustrates use of formulas. Method applies to continuous as well as simply supported shells.**

# Cylindrical Shell Analysis Simplified by Beam Method

By JAMES CHINN

■ THE DESIGN OF CYLINDRICAL SHELLS has been considerably simplified of late by the publication of ASCE Manual No. 31, *Design of Cylindrical Concrete Shell Roofs*. Coefficients are given for the calculation of shells having various ratios of $r/t$ and $l/r$. Shells having other ratios of $r/t$ and $l/r$ can be calculated using formulas given; however, the calculation requires the solution of four simultaneous equations.

Lundgren's[1] "beam method," which involves no more than $Mc/I$, $VQ/Ib$, and normal indeterminate structures analysis, can be used with sufficient accuracy on shells with certain edge conditions without resorting to simultaneous equations. The single shell with not-too-deep edge beams[2] can be designed by this method, as can single shells without edge beams having $l/r$ greater than about 5.[3] Interior barrels of multiple shells can also be designed by this method, and the results for a shell with $l/r = 2$ and for a shell with $l/r = 1.11$ are presented and compared to the results obtained by using Manual 31.

## EXPLANATION OF BEAM METHOD

Assumptions involved in the beam method, aside from the usual assumptions of the flexure formula are:

1. All points on a transverse cross section deflect the same amount vertically and do not deflect horizontally.
2. No radial shear, radial bending stresses, or twisting stresses act on transverse cross sections.

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*The assumptions made also constitute the limitations of the method, and when the assumptions do not agree closely with the true action of the shell, this method cannot be applied.*

The beam method involves two calculations: the beam calculation and the arch calculation. In the beam calculation, the longitudinal stresses and the membrane shear stresses are calculated using the beam flexure formula, $Mc/I$, and the beam shear formula, $VQ/Ib$, respectively (Fig. 1).

The arch calculation is a bit more complicated. An element of length $dx$ is taken from the shell, and the forces acting on it in transverse directions are considered. These consist of the external load, the membrane shear stresses, and, in the case of the multiple shell, the edge moment and the edge force (Fig. 2). The vertical support for the "arch" is provided by the vertical component of the shear-difference stresses $dV \cdot Q/Ib$, rather than by vertical reactions at the springing as in the more usual arch. These stresses, acting tangent to the arch, produce transverse moments as well as do the external loads. In solving for these two quantities, simultaneous equations can be avoided if reactions are determined at the elastic center rather than at the edge.

For the case of the shell with edge beams, Lundgren used a method in which the shell is divided into segments and the distributed loads are replaced by concentrated loads at the midpoints of these segments.[4] This method is also applicable to the single shell without edge beams and the interior barrel of a multiple shell, but a more exact solution is possible.

Lundgren gave approximate formulas for final transverse moments caused by load uniformly distributed on the horizontal projection of the single shell without edge beam and on the interior barrel of a multiple shell.[5] The equations which follow have been developed by the writer and are exact insofar

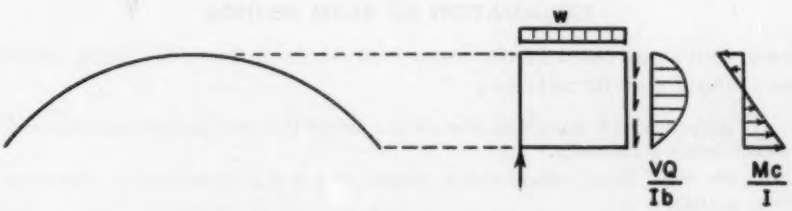
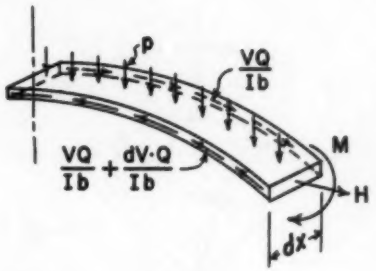



Fig. 1—Longitudinal stress and transverse shear stress distributions

Fig. 2—Forces acting on "arch"




as the assumptions involved in the beam method are exact. These equations require a high degree of accuracy in numerical calculation and are therefore not suited to slide rule calculation.

## NOTATION

$b$ = thickness of section where transverse shear stress is calculated
$c$ = distance from neutral axis to extreme fiber
$E$ = modulus of elasticity
$F_h$ = horizontal component of force
$F_v$ = vertical component of force
$H_{EC}$ = horizontal reaction at elastic center
$I$ = moment of inertia of circular arc about neutral axis
$l$ = longitudinal span
$M$ = moment
$M_{EC}$ = moment at elastic center
$M\phi$ = moment acting on arch
$Q$ = first moment about neutral axis of area of cross section above (or below) section where transverse shear stress is calculated
$r$ = radius to center of shell
$S$ = tangential shearing stress ($VQ/Ib$)
$t$ = thickness of shell
$T_X$ = longitudinal stress ($Mc/I$)
$T\phi$ = direct stress acting on arch
$V$ = beam shear force
$(\Delta_h)_{EC}$ = horizontal deflection at elastic center
$(\theta)_{EC}$ = rotation at elastic center
$\phi$ = angle measured from vertical
$\phi_k$ = angle from vertical to edge of shell

## EQUATIONS FOR FORCES AND DEFORMATIONS

The following equations have been developed by the writer using the free bodies in Fig. 3. $F_v$ was obtained by summing vertical forces, $F_h$ by summing horizontal forces, and $M$ by taking moments about the intersection of the $F_v$ force and the arc of the shell. Integration was used when required.

$(Et^3/12)\,\Theta_{EC}$ was obtained as the integral $\int_0^{\phi_k} Mrd\phi$, and $(Et^3/12)\,(\Delta_h)_{EC}$ was obtained as the integral

$$\int_0^{\phi_k} Myrd\phi = \int_0^{\phi_k} Mrd\phi \left[ r\cos\phi - \frac{r\sin\phi_k}{\phi_k} \right]$$

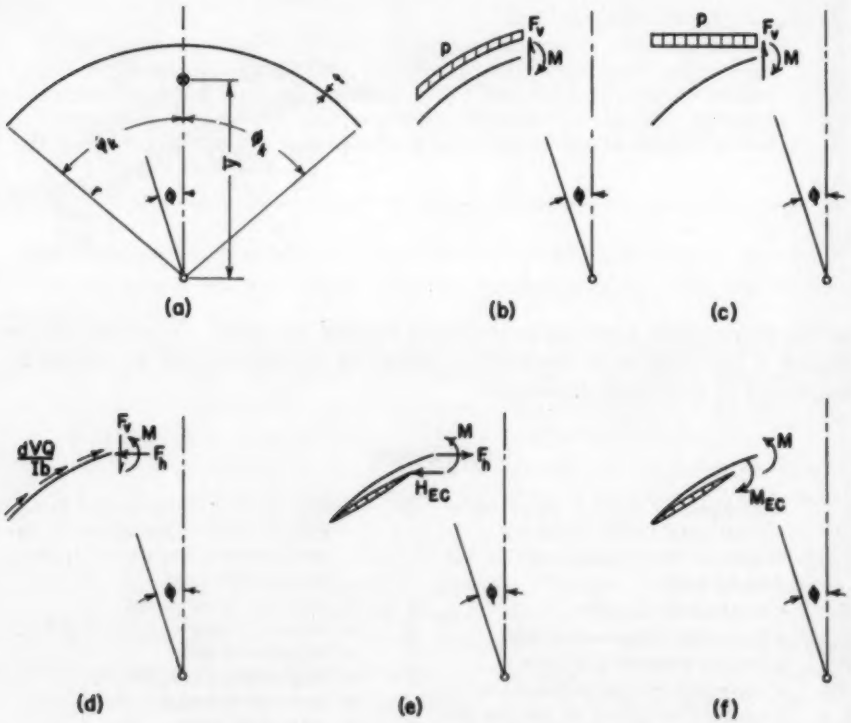



Fig. 3—Properties of arch cross section and component loads

**Properties of cross section (Fig. 3a)**

$$y = \frac{r \sin \phi_k}{\phi_k} \quad \ldots\ldots (1)$$

$$I = r^3 \left[ \phi_k + (\sin \phi_k) \left( \cos \phi_k - 2 \frac{\sin \phi_k}{\phi_k} \right) \right] \quad \ldots\ldots (2)$$

$$Q = 2r^2 \left( \sin \phi - \frac{\phi}{\phi_k} \sin \phi_k \right) \quad \ldots\ldots (3)$$

**Dead load (Fig. 3b)**

$$F_r = pr (\phi_k - \phi) \quad \ldots\ldots (4)$$

$$M = pr^2 [\cos \phi - \cos \phi_k + (\sin \phi) (\phi - \phi_k)] \quad \ldots\ldots (5)$$

$$\frac{Et^3}{12} \theta_{EC} = pr^3 [2 \sin \phi_k - \phi_k (\cos \phi_k + 1)] \quad \ldots\ldots (6)$$

$$\frac{Et^3}{12}(\Delta_h)_{EC} = pr^4\left[\frac{\phi_k}{4} + \frac{3}{8}\sin 2\phi_k + \sin\phi_k - 2\frac{\sin^3\phi_k}{\phi_k}\right] \quad \ldots\ldots(7)$$

**Uniform load (Fig. 3c)**

$$F_v = pr(\sin\phi_k - \sin\phi) \quad \ldots\ldots(8)$$

$$M = \frac{pr^2}{2}(\sin\phi_k - \sin\phi)^2$$

$$= \frac{pr^2}{2}(\sin^2\phi_k - 2\sin\phi_k\sin\phi + \sin^2\phi) \quad \ldots\ldots(9)$$

$$\frac{Et^3}{12}\theta_{EC} = \frac{pr^3}{2}\left[\phi_k\sin^3\phi_k + \frac{3}{4}\sin 2\phi_k - 2\sin\phi_k + \frac{\phi_k}{2}\right] \quad \ldots\ldots(10)$$

$$\frac{Et^3}{12}(\Delta_h)_{EC} = \frac{pr^4}{2}\left[\frac{\sin^2\phi_k}{\phi_k}\left(2 - \frac{3}{2}\cos\phi_k\right) - \frac{2}{3}\sin^3\phi_k - \frac{\sin\phi_k}{2}\right] \quad \ldots\ldots(11)$$

**Shear difference (Fig. 3d)**

$$F_v = \frac{wr^3}{I}\left[\frac{\phi_k - \phi}{2} + \frac{1}{4}(\sin 2\phi_k + \sin 2\phi)\right.$$

$$\left. - \left(\frac{\sin\phi_k}{\phi_k}\right)(\sin\phi_k - \sin\phi + \phi\cos\phi)\right] \quad \ldots\ldots(12)$$

$$F_h = \frac{wr^3}{I}\left[\frac{\sin^2\phi_k + \sin^2\phi}{2} + \frac{\sin\phi_k}{\phi_k}(\cos\phi_k - \cos\phi - \phi\sin\phi)\right] \quad \ldots\ldots(13)$$

$$M = \frac{wr^4}{I}\left[\cos\phi - \cos\phi_k + \frac{\sin\phi_k}{2\phi_k}\phi^2 - \frac{\phi_k}{2}(\sin\phi_k + \sin\phi)\right.$$

$$+ \frac{\phi\sin\phi}{2} + \frac{\sin^2\phi_k}{2}\cos\phi - \frac{\sin 2\phi_k}{4}\sin\phi + \frac{\sin^2\phi_k}{\phi_k}\sin\phi$$

$$\left. + \frac{\sin 2\phi_k}{2\phi_k}\cos\phi - \frac{\sin\phi_k}{\phi_k}\right] \quad \ldots\ldots(14)$$

$$\frac{Et^3}{12}\theta_{EC} = \frac{wr^3}{I}\left[\frac{\sin\phi_k}{2} - \phi_k\left(\cos\phi_k + \frac{1}{2}\right) - \frac{\phi_k^2\sin\phi_k}{3} + \frac{\sin^3\phi_k}{2}\right.$$

$$\left. + \frac{\sin^2\phi_k}{\phi_k} + \frac{\sin 2\phi_k}{4}(\cos\phi_k - 1)\right] \quad \ldots\ldots(15)$$

$$\frac{Et^3}{12}(\Delta_h)_{EC} = \frac{wr^4}{I}\left[\frac{3}{8}\phi_k + \frac{17}{16}\sin 2\phi_k + \frac{7}{12}\phi_k\sin^2\phi_k + \frac{\sin\phi_k}{2}\right.$$

$$\left. - \frac{\sin^2\phi_k}{\phi_k}\left(2.5 - \frac{\cos\phi_k}{2} + \frac{\sin\phi_k}{\phi_k}\right)\right] \quad \ldots\ldots(16)$$

**H at elastic center (Fig. 3e)**

$$M = H_{EC} r \left( \cos \phi - \frac{\sin \phi_k}{\phi_k} \right) \quad (17)$$

$$\frac{Et^3}{12} \theta_{EC} = 0 \quad (18)$$

$$\frac{Et^3}{12} (\Delta_k)_{EC} = \frac{I}{2} H_{EC} \quad (19)$$

**M at elastic center (Fig. 3f)**

$$\frac{Et^3}{12} \theta_{EC} = r \phi_k M_{EC} \quad (20)$$

$$\frac{Et^3}{12} (\Delta_k)_{EC} = 0 \quad (21)$$

## NUMERICAL EXAMPLE

The use of these equations will be illustrated by calculating Example 2 from ASCE Manual No. 31,[2] a shell having $l/r = 2$ (see Fig. 4).

### Beam calculation

Using Eq. (1), the neutral axis is found to lie 28.543 ft from the center of the circle, making the $c$ distances to the top and bottom fibers 2.457 ft and 4.796 ft, respectively. By Eq. (2), $I = 204.777$ ft$^3$. Applying $T_X = Mc/I$, the top and bottom fiber stresses at the middle of the shell are 17.470 and 34.101 kips per ft, respectively. The variation of $T_X$ by the beam method is plotted against the values from Manual No. 31 in Fig. 5. The values from the manual have been multiplied by 0.125/0.128 because the manual values were calculated by replacing the actual load with a load which varied as a half sine wave in the longitudinal direction. The ratio of the moment due to uniform load to the moment due to the sinusoidal load is 0.125/0.128.

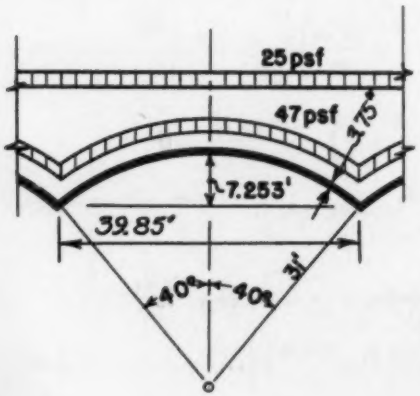
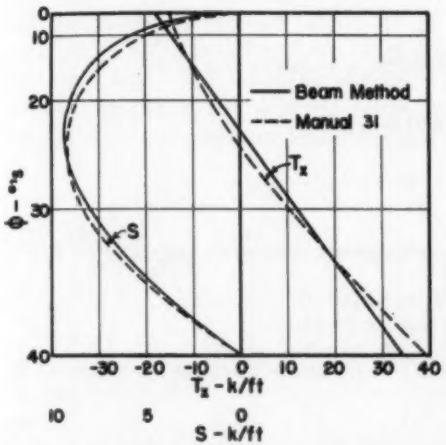



Fig. 4—Dimensions and loads, shell of Example 2, ASCE Manual No. 31. Simply supported, longitudinal span = 62 ft. $l/r = 2$



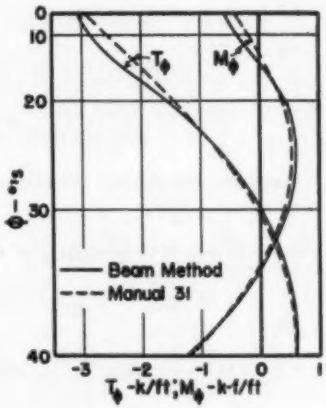




Fig. 5—(left)—Comparison of $T_X$ and $S$ using Example 2 of ASCE Manual No. 31
Fig. 6 (right)—Comparison of $T\phi$ and $M\phi$ using shell of Example 2, ASCE Manual No. 31

Values of $Q/2$ and of $S = VQ/2I$ calculated at 10-deg intervals are shown in Table 1 and plotted against the values from Manual No. 31 in Fig. 5. The values from the manual have been multiplied by 0.500/0.405 to compensate for the sinusoidal load.

TABLE 1—BEAM SHEAR STRESSES

| $\phi$, deg | $Q/2$, ft³ | $S = \frac{VQ}{2I}$, kips per ft |
|---|---|---|
| 0 | 0 | 0 |
| 10 | 24.89 | 5709 |
| 20 | 39.65 | 9092 |
| 30 | 34.42 | 7894 |
| 40 | 0 | 0 |

## Arch calculation

Numerical substitution in the equations for $(Et^3/12)\,\theta_{EC}$ yields:

$$\text{Eq. (6)} \quad \frac{Et^3}{12}\theta_{EC} = 47\,(31)^3\,(1.2855752 - 1.2329316) = 73{,}710.36$$

$$\text{Eq. (10)} \quad \frac{Et^3}{12}\theta_{EC} = \frac{25}{2}\,(31)^3\,(0.2884512 + 0.7386059 - 1.2855752 + 0.3490659) = 33{,}718.87$$

$$\text{Eq. (15)} \quad \frac{Et^3}{12}\theta_{EC} = \frac{3.030\,(31)^5}{204.777}\,(0.3213938 - 0.8838657 - 0.1044289 + 0.1327922 - 0.0576003 + 0.5918309) = 51{,}685.93$$

$$\text{Eq. (20)} \quad \frac{Et^3}{12}\theta_{EC} = 31\,(0.698132)\,M_{EC} = 21.642092\,M_{EC}$$

Solving for $M_{EC}$,

$$73{,}710.36 + 33{,}718.87 - 51{,}685.93 - 21.642092\, M_{EC} = 0$$

$$M_{EC} = 2575.69 \text{ ft-lb per ft}$$

Numerical substitution in the equations for $(Et^3/12)\,(\Delta_h)_{EC}$ yields:

$$\text{Eq. (7)} \quad \frac{Et^3}{12}(\Delta_h)_{EC} = 47\,(31)^4\,(0.1745329 + 0.3693029 + 0.6427876 - 1.1836617) = 128{,}554.03$$

$$\text{Eq. (11)} \quad \frac{Et^3}{12}(\Delta_h)_{EC} = \frac{25}{2}\,(31)^4\,(0.5036087 - 0.1770562 - 0.3213938) = 59{,}552.10$$

$$\text{Eq. (16)} \quad \frac{Et^3}{12}(\Delta_h)_{EC} = \frac{3.030\,(31)^6}{204.777}\,[0.2617994 + 1.0463583 + 0.16826320 + 0.3213938 - 0.5918309\,(2.5 - 0.3830222 + 0.9207254)] = 106{,}379.81$$

$$\text{Eq. (19)} \quad \frac{Et^3}{12}(\Delta_h)_{EC} = \frac{204.777}{2}\,H_{EC} = 102.389\,H_{EC}$$

Solving for $H_{EC}$,

$$128{,}554.03 + 59{,}552.10 - 106{,}379.81 - 102.389\,H_{EC} = 0$$

$$H_{EC} = 798.19 \text{ lb per ft}$$

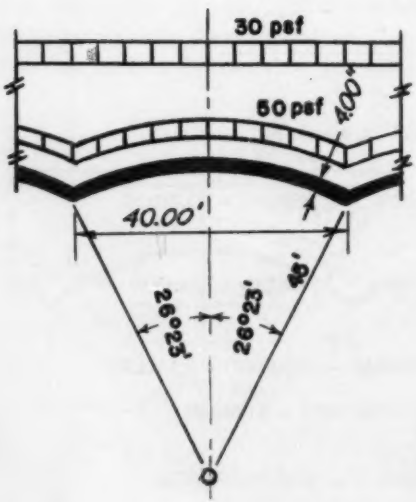
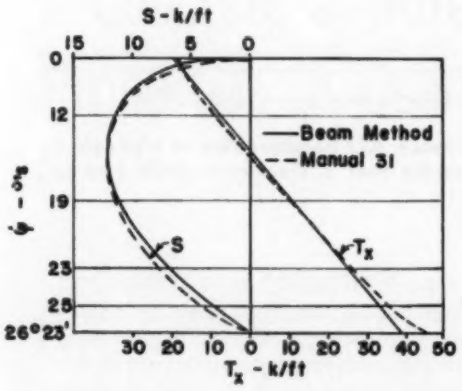



Fig. 7—Dimension and loads of shell from PCA's ST 77. Simply supported, longitudinal span = 50 ft. $l/r$ = 1.11

Final moments $M\phi$ are calculated at 10-deg intervals using Eq. (5), (9), (14), and (17). Final direct stresses $T\phi$ are calculated using Eq. (4), (8), (12), (13), and $H_{EC}$. A comparison of results by the beam method and by manual No. 31 is given in Fig. 6. The values from the manual have been multiplied by $\pi/4$ to compensate for the sinusoidal load assumed therein.



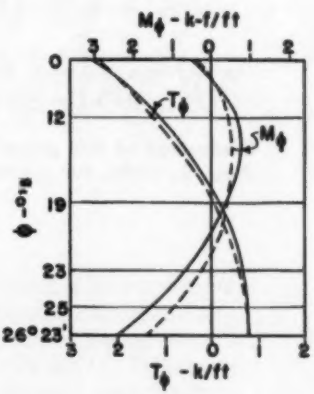




Fig. 8 (left)—Comparison of $T_X$ and $S$ for shell of PCA's ST 77. Fig. 9 (right)—Comparison of $T\phi$ and $M\phi$ for shell of PCA's ST 77

Calculations by the beam method are here omitted, but results for the shell of the Portland Cement Association *Bulletin* ST 77, "Design of Barrel Shell Roofs," are presented in Fig. 7, 8, and 9. This shell has an $l/r = 1.11$, and the results by Manual No. 31 are shown for comparison.

## SUMMARY

The beam method for analyzing cylindrical shells can be used with sufficient accuracy in many practical cases. It can be used for any values of $r/t$ and $l/r$ *provided the assumptions involved in the method are sufficiently satisfied.* A particular advantage is that it can also be used under all conditions of support, and no special equations are required for the cases of overhangs and continuity. The equations given above facilitate the calculation of single shells without edge beams and of interior barrels of multiple shells.

## ACKNOWLEDGMENT

The writer wishes to express his thanks to Eliot J. Bernstein, Associated Specialties Co., Salt Lake City, for performing the beam method calculation for the shell with $l/r = 1.11$.

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2. *Design of Cylindrical Concrete Shell Roofs*, ASCE Manual No. 31, American Society of Civil Engineers, New York, 1956, pp. 58-63.

3. *Ibid.*, p. 2.

4. Lundgren, *op. cit.*, pp. 73-76.

5. Lundgren, *op. cit.*, pp. 72-73.

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Title No. 55-72

# Flexural and Compressive Strength of Concrete as Affected by the Properties of Coarse Aggregates

By M. F. KAPLAN

EXPERIMENTS WERE CONDUCTED to determine the effects of the properties of 13 different coarse aggregates on the flexural and compressive strength of concrete. Depending on the aggregate, differences of 40 percent in flexural strength and 29 percent in compressive strength were obtained for concrete of the same mix proportions. Statistical analysis indicated that the shape, surface texture, and modulus of elasticity of the aggregates were the main causes of variation in concrete strength. The greater the strength of the concrete, the more important these effects became. The elastic modulus of the aggregate was, in general, the most important single factor affecting flexural strength, although for concrete with the greatest strength, surface texture had the predominating effect. Surface texture was the most important aggregate property influencing concrete compressive strength. Because the aggregate strength was generally greater than that of the concrete and differences in the water-absorptive capacities of the aggregates were small, no relationship was found between the strength or water-absorptive capacity of the aggregates used and the strength of the concrete. However, this should not be assumed as evidence that coarse aggregates of low strength or high water-absorptive capacity will not affect concrete strength.

CONCRETE OF GIVEN MIX PROPORTIONS may have different strengths when different types of coarse aggregate are used. For example, Wright[1] has shown that a certain crushed rock aggregate gave concrete of higher flexural strength than a rounded gravel aggregate. Similar effects were noted in regard to compressive strength.[2] The surface texture, shape, strength, chemical composition, elastic properties, and water-absorption capacity of the aggregate have all been suggested[3-12] as possible causes of the variations in the strength of concrete of the same mix proportions. However, little has been done to assess quantitatively the effects of these properties of coarse aggregates on the strength of concrete.

## DETAILS OF THE TESTING PROGRAM

Concrete specimens were made with 13 different coarse aggregates having a wide range of physical properties (Table 1). The fine aggregate in all concrete mixes was Chertsey river sand. All aggregates were dried before use and ordinary portland cement taken from a single batch, as manufactured, was thoroughly mixed and used for all concrete specimens. Each of the 13 coarse aggregates was used in the three mixes shown in Table 2.

Aggregate grading curves are shown in Fig. 1, and the yield, cement factor, and workability of the concretes are given in Table 3. Workability was determined by the compacting factor method;[14] approximate equivalents in terms of slump are given at the bottom of Table 3. Effects of the aggregates on workability of concrete have been fully discussed by the author in another paper.[15]

### Aggregate properties

Flakiness, elongation, crushing value, impact value, and crushing strength were tested in accordance with the relevant sections of British Standard 812.[16]

*Flakiness index*—The flakiness of the aggregate is measured by expressing the total weight of different size fractions of the aggregate, passing specified thickness gages, as a percentage of the total weight of the sample tested. The width of the thickness gages is 0.6 times the mean sieve size of the aggregate.

*Elongation index*—The same as for flakiness except that length gages equal to 1.8 times the mean sieve size of the aggregate are used.

*Aggregate crushing value*—This is obtained by subjecting a stipulated quantity of the ½-in. to ⅜-in. fraction of the coarse aggregate to a specified compressive load. The weight of aggregate passing a No. 7 British Standard (No. 8 Tyler) sieve is then expressed as a percentage of the total. This percentage is the aggregate crushing value.

Water absorption and specific gravity were measured according to Shergold;[18] angularity was also evaluated by Shergold's method.[17] The degree of angularity is determined by measuring the proportion of voids in the ¾-to ½-in. fraction of the aggregate when compacted in a prescribed way. The "angularity number" is the percentage of voids less 33.

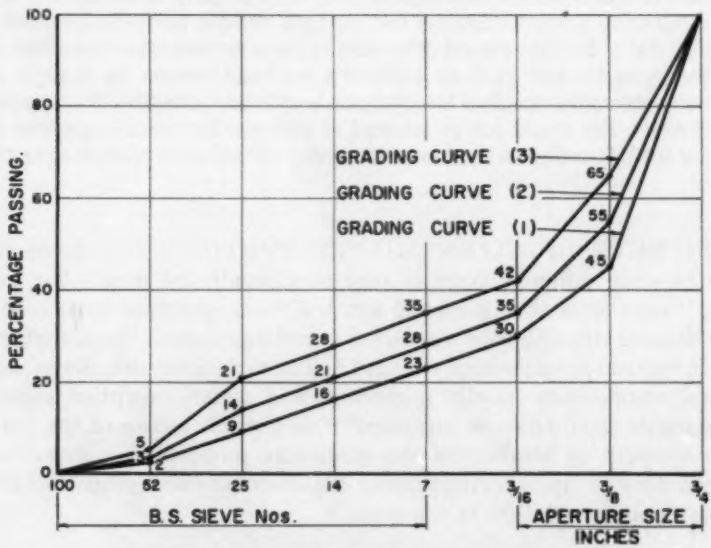


Fig. 1—The grading of the aggregates

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Surface texture was determined according to the method of Wright.[19] This involves embedding a particle of aggregate in a synthetic resin and obtaining thin sections of the aggregate surrounded by the resin. The interface between stone and resin is magnified 125 times on a projection microscope and traced. The length of the profile is measured and compared with the length of an uneveness line drawn as a series of chords. The difference between the two lengths is taken as a measure of the roughness or surface texture of the aggregate.

Flexural and tensile strength were measured according to Hosking.[20] Dynamic modulus of elasticity, ultrasonic pulse velocity, and Poisson's ratio were tested by the methods followed by Hardman.[21]

Test results are given in Table 4. The crushing, flexural, and tensile strengths of the Bridport and Chertsey gravels (A, B, C, D, and E) were not obtained, due to the frequency of breakage when drilling the 1-in. cylindrical cores. The elastic properties of these aggregates were determined by tests on large pieces of the aggregate, approximately 7 in. long and having a thickness of not more than one-fifth of the length.

## Concrete strength

Concrete beams 4 in. square and 20 in. long were made in accordance with British Standard 1881.[14] The flexural strength and compressive strength were determined in accordance with Parts 11 and 9, respectively, of British Standard 1881. Three beam specimens of each mix were tested at 7, 28, and 91 days, respectively, and the results are given in Tables 5 and 6. Each of the three test specimens for each mix tested at a given age, was made from different batches of concrete mixed on different days.

Mortar beams were also made in which the cement:sand:water proportions by weight were the same as in the concrete mixes, i.e., 1:0.92:0.35, 1:2.6:0.60, and 1:4.3:0.85 for Mixes I, II, and III, respectively, and their flexural and compressive strengths determined. The results, each of which is the average of tests on three beams, are given in Table 7.

**TABLE 1—TYPE, SOURCE, AND CONDITION OF THE COARSE AGGREGATES USED**

| Aggregate identification | Type | Source in United Kingdom | Condition |
|---|---|---|---|
| A | Quartzite gravel | Bridport | Natural |
| B | Quartzite gravel | Bridport | Crushed with a low reduction ratio from 1½-in. single size gravel |
| C | Flint gravel | Chertsey | Natural |
| D | Flint gravel | Chertsey | Crushed with a low reduction ratio from 1½-in. single size gravel for the ¾-in. to ⅜-in. fraction and from ¾-in. single size gravel for the ⅜-in. to ⅛-in. fraction |
| E | Flint gravel | Chertsey | Crushed with a high reduction ratio from 2-in. to 3-in. size natural gravel |
| F | Limestone gravel | Lancashire | Natural |
| G | Limestone gravel | Lancashire | Crushed with a low reduction ratio from 1½-in. single size gravel |
| H | Basalt | Blodwell | Crushed with a low reduction ratio from 1½-in. single size |
| J | Basalt | Blodwell | Crushed with a high reduction ratio from 3-in. to 4-in. size |
| K | Granite | Penryn | Crushed with a low reduction ratio from 1½-in. single size |
| L | Granite | Mountsorrel | Crushed with a low reduction ratio from 1½-in. single size |
| M | Trachyte | Downhead | Crushed with a low reduction ratio from 1½-in. single size |
| N | Limestone | Somerset | Quarried and crushed by supplier |

## TABLE 2—THE THREE MIXES USED FOR THE TEST PROGRAM

| Mix | Cement-aggregate ratio (by weight) | Water-cement ratio (by weight) | Aggregate grading |
|---|---|---|---|
| I | 1:3.08 | 0.35 | Grading curve No. 1 *Road Note* No. 4[13] |
| II | 1:7.53 | 0.60 | Grading curve No. 2 *Road Note* No. 4 |
| III | 1:10.25 | 0.85 | Grading curve No. 3 *Road Note* No. 4 |

## TABLE 3—YIELD, CEMENT FACTOR, AND WORKABILITY OF THE CONCRETE

| Aggregate | Mix I | | Mix II | | | Mix III | | |
|---|---|---|---|---|---|---|---|---|
| | Yield, cu ft per 94-lb sack | Cement factor, sacks per cu yd | Yield, cu ft per 94-lb sack | Cement factor, sacks per cu yd | Compacting factor* | Yield, cu ft per 94-lb sack | Cement factor, sacks per cu yd | Compacting factor* |
| A | 2.87 | 9.42 | 5.90 | 4.58 | 0.91 | 7.86 | 3.43 | 0.89 |
| B | 2.86 | 9.45 | 5.89 | 4.59 | 0.81 | 7.85 | 3.44 | 0.82 |
| C | 2.86 | 9.45 | 5.89 | 4.59 | 0.81 | 7.85 | 3.44 | 0.82 |
| D | 2.85 | 9.48 | 5.85 | 4.62 | 0.81 | 7.80 | 3.46 | 0.80 |
| E | 2.83 | 9.54 | 5.83 | 4.64 | 0.80 | 7.75 | 3.49 | 0.81 |
| F | 2.82 | 9.58 | 5.79 | 4.67 | 0.88 | 7.73 | 3.50 | 0.88 |
| G | 2.81 | 9.62 | 5.79 | 4.67 | 0.84 | 7.71 | 3.51 | 0.83 |
| H | 2.80 | 9.65 | 5.75 | 4.70 | 0.81 | 7.70 | 3.51 | 0.81 |
| J | 2.80 | 9.65 | 5.75 | 4.70 | 0.80 | 7.70 | 3.51 | 0.79 |
| K | 2.81 | 9.62 | 5.79 | 4.67 | 0.81 | 7.71 | 3.51 | 0.81 |
| L | 2.82 | 9.58 | 5.79 | 4.67 | 0.81 | 7.73 | 3.50 | 0.80 |
| M | 2.77 | 9.76 | 5.68 | 4.76 | 0.84 | 7.60 | 3.55 | 0.80 |
| N | 2.79 | 9.68 | 5.74 | 4.71 | 0.86 | 7.66 | 3.53 | 0.85 |

*Compacting factor of 0.78 corresponds to slump of 0 to 1 in.
Compacting factor of 0.85 corresponds to slump of 1 to 2 in.
Compacting factor of 0.92 corresponds to slump of 2 to 4 in.

## TABLE 4—PROPERTIES OF THE AGGREGATES

| Aggregate | Shape | | | Strength | | | | | Elastic properties | | | Surface texture (roughness factor) | Absorptive capacity, percent | Specific gravity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Average flakiness index | Average elongation index | Angularity No. | Crushing, psi | Flexural, psi | Indirect tensile, psi | Crushing value | Impact value | Dynamic modulus × $10^6$ psi | Pulse velocity, ft per sec | Poisson's ratio | | | |
| A | 8 | 24 | 1 | — | — | — | 8 | 15 | 8.6 | 16,370 | 0.15 | 4.4 | 0.5 | 2.51 |
| B | 27 | 35 | 5 | — | — | — | 16 | 20 | 8.6 | 16,370 | 0.15 | 5.3 | 0.4 | 2.52 |
| C | 28 | 33 | 6 | — | — | — | 16 | 22 | 8.0 | 16,640 | 0.24 | 2.5 | 1.0 | 2.52 |
| D | 22 | 27 | 8 | — | — | — | 20 | 23 | 8.0 | 16,640 | 0.24 | 2.2 | 0.3 | 2.55 |
| E | 34 | 42 | 9 | — | — | — | 20 | 28 | 8.0 | 16,640 | 0.24 | 2.2 | 0.1 | 2.58 |
| F | 7 | 13 | 2 | 22,900 | 2720 | 1100 | 18 | 24 | 10.7 | 21,100 | 0.33 | 9.4 | 0.9 | 2.61 |
| G | 22 | 33 | 6 | 22,900 | 2720 | 1100 | 23 | 23 | 10.7 | 21,100 | 0.33 | 7.5 | 0.8 | 2.62 |
| H | 31 | 42 | 9 | 44,500 | 6490 | 2210 | 11 | 11 | 10.9 | 19,830 | 0.29 | 13.1 | 0.6 | 2.64 |
| J | 34 | 44 | 10 | 44,500 | 6490 | 2210 | 14 | 14 | 10.9 | 19,830 | 0.29 | 12.2 | 0.5 | 2.64 |
| K | 9 | 18 | 9 | 22,950 | 900 | 920 | 29 | 33 | 3.3 | 12,780 | 0.36 | 16.6 | 0.3 | 2.62 |
| L | 23 | 30 | 8 | 35,800 | 3650 | 1690 | 19 | 21 | 10.2 | 18,950 | 0.27 | 8.8 | 0.2 | 2.61 |
| M | 25 | 43 | 10 | 41,400 | 5060 | 2400 | 12 | 11 | 10.5 | 19,330 | 0.30 | 11.6 | 0.2 | 2.71 |
| N | 26 | 33 | 7 | 26,700 | 2270 | 1230 | 20 | 20 | 11.3 | 21,080 | 0.31 | 10.0 | 0.4 | 2.66 |

## TABLE 5—FLEXURAL STRENGTH OF THE CONCRETE, PSI

| Aggregate | Mix I | | | Mix II | | | Mix III | | |
|---|---|---|---|---|---|---|---|---|---|
| | 7 days | 28 days | 91 days | 7 days | 28 days | 91 days | 7 days | 28 days | 91 days |
| A | 670 | 775 | 840 | 435 | 530 | 560 | 280 | 365 | 445 |
| B | 750 | 855 | 975 | 495 | 630 | 705 | 280 | 420 | 500 |
| C | 650 | 755 | 740 | 445 | 605 | 590 | 335 | 425 | 475 |
| D | 735 | 825 | 845 | 525 | 650 | 675 | 300 | 415 | 525 |
| E | — | 870 | 870 | — | 660 | 710 | — | 385 | 415 |
| F | 760 | 825 | 835 | 520 | 655 | 690 | 315 | 430 | 450 |
| G | 775 | 950 | 950 | 535 | 695 | 760 | 320 | 480 | 550 |
| H | 805 | 1000 | 960 | 535 | 740 | 770 | 350 | 480 | 555 |
| J | 830 | 1060 | 1045 | 540 | 725 | 840 | 335 | 490 | 560 |
| K | 685 | 820 | 940 | 470 | 575 | 635 | 290 | 390 | 455 |
| L | 775 | 940 | 955 | 515 | 695 | 755 | 335 | 470 | 520 |
| M | 755 | 970 | 1015 | 540 | 695 | 820 | 315 | 445 | 515 |
| N | 850 | 975 | 1060 | 555 | 795 | 805 | 330 | 480 | 540 |

## TABLE 6—COMPRESSIVE STRENGTH OF THE CONCRETE, PSI

| Aggregate | Mix I | | | Mix II | | | Mix III | | |
|---|---|---|---|---|---|---|---|---|---|
| | 7 days | 28 days | 91 days | 7 days | 28 days | 91 days | 7 days | 28 days | 91 days |
| A | 7780 | 9770 | 10360 | 3770 | 5910 | 6880 | 1880 | 3140 | 4010 |
| B | 8080 | 10500 | 11640 | 4100 | 6240 | 7380 | 2140 | 3330 | 4140 |
| C | 8160 | 9520 | 10020 | 4460 | 6660 | 7470 | 2260 | 3610 | 4420 |
| D | 8100 | 9960 | 10850 | 4360 | 6700 | 7800 | 2140 | 3540 | 4340 |
| E | — | 9990 | 10420 | — | 5480 | 6920 | — | 2840 | 3530 |
| F | 8040 | 10180 | 11170 | 4420 | 6660 | 7690 | 2170 | 3370 | 4360 |
| G | 8720 | 10160 | 11440 | 4300 | 6440 | 7450 | 2230 | 3520 | 4360 |
| H | 8440 | 11130 | 11280 | 4550 | 6910 | 8050 | 2420 | 3660 | 4570 |
| J | 8670 | 10950 | 11770 | 4490 | 6600 | 7850 | 2420 | 3600 | 4420 |
| K | 8290 | 10620 | 11270 | 4020 | 6100 | 7290 | 2150 | 3310 | 4130 |
| L | 8940 | 11240 | 12430 | 4460 | 6720 | 8020 | 2280 | 3820 | 4650 |
| M | 8680 | 11460 | 12240 | 3850 | 6150 | 7450 | 2180 | 3260 | 4180 |
| N | 8140 | 10410 | 11460 | 4250 | 6500 | 7500 | 2170 | 3340 | 4070 |

## TABLE 7—FLEXURAL AND COMPRESSIVE STRENGTH OF THE MORTAR IN THE CONCRETE AFTER 91 DAYS

| Property | Mix I | Mix II | Mix III |
|---|---|---|---|
| Flexural strength, psi | 1130 | 795 | 525 |
| Compressive strength, psi | 11090 | 5920 | 3380 |

## TABLE 8—VARIATIONS IN THE FLEXURAL AND COMPRESSIVE STRENGTH OF CONCRETE WHEN USING DIFFERENT AGGREGATES

| Property | Mix I | | | Mix II | | | Mix III | | |
|---|---|---|---|---|---|---|---|---|---|
| | 7 days | 28 days | 91 days | 7 days | 28 days | 91 days | 7 days | 28 days | 91 days |
| Flexural strength, psi | | | | | | | | | |
| Highest | 850 | 1060 | 1060 | 555 | 795 | 840 | 350 | 490 | 560 |
| Lowest | 650 | 755 | 740 | 435 | 530 | 560 | 280 | 365 | 415 |
| Highest minus lowest | 200 | 305 | 320 | 120 | 265 | 280 | 70 | 125 | 145 |
| Average | 754 | 893 | 925 | 508 | 665 | 717 | 315 | 436 | 499 |
| *Range, percent | 27 | 34 | 34 | 23 | 40 | 39 | 22 | 28 | 29 |
| Compressive strength, psi | | | | | | | | | |
| Highest | 8940 | 11460 | 12430 | 4550 | 6910 | 8050 | 2420 | 3820 | 4650 |
| Lowest | 7780 | 9520 | 10020 | 3770 | 5480 | 6880 | 1880 | 2840 | 3530 |
| Highest minus lowest | 1160 | 940 | 2410 | 780 | 1430 | 1170 | 540 | 980 | 1120 |
| Average | 8340 | 10450 | 11260 | 4250 | 6380 | 7520 | 2200 | 3410 | 4240 |
| *Range, percent | 14 | 19 | 21 | 18 | 22 | 16 | 24 | 29 | 26 |

*Range, percent $= \frac{\text{highest} - \text{lowest}}{\text{average}} \times 100$

## DISCUSSION OF RESULTS

### Properties of the aggregates

The flexural strength or modulus of rupture of the aggregates was generally between two and three times as great as the tensile strength obtained from the indirect or "Brazilian" tensile strength test. There was, however, good correlation between the results of the two tests, the correlation coefficient being +0.93. The flexural strength of the Penryn granite (K) was slightly lower than its indirect tensile strength, a result which is believed to be due to planes of weakness in this aggregate which cause low flexural strength.

As shown in Fig. 2, the crushing, flexural, and tensile strengths of the aggregates are related to their "crushing value," the correlation coefficients being −0.84, −0.94, and −0.89, respectively. There was also good correlation (+0.88) between the crushing and impact values.

There was no evidence of a relationship between the strength and dynamic modulus of elasticity of the different aggregates. This confirms Hosking's[20] finding that, although there are indications of a relationship between strength and elastic properties for different samples of the same aggregate, no such

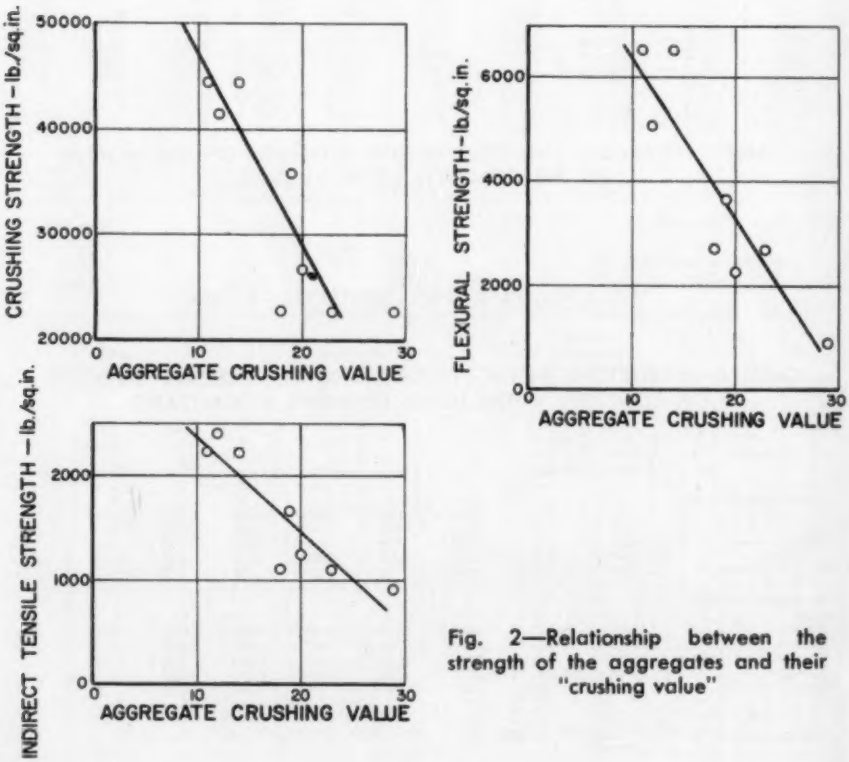


Fig. 2—Relationship between the strength of the aggregates and their "crushing value"

relationship could be established for different types of aggregate. Ultrasonic pulse velocity and dynamic modulus were, however, related, the correlation coefficient being + 0.95.

#### Strength properties of the concrete

As can be seen from the values given in Table 8, concrete of the same mix proportions may have different flexural and compressive strengths, depending on the type of aggregate used in the mix.

The maximum range in the flexural and compressive strengths for the same mix proportions, at the same age, was 40 and 29 percent, respectively. The average range was 31 and 21 percent, respectively. The effects of different aggregates on compressive strength were therefore not as great as on the flexural strength of concrete. Nevertheless, the effects were considerable; by using a selected aggregate in Mix I, it was possible to obtain an increase of 2410 psi. and 320 psi, respectively, in the compressive and flexural strengths, after 91 days, for concrete of the same mix proportions.

#### Effects of aggregate properties on concrete strength

To obtain some indication as to which properties of the aggregate influenced the strength of the concrete, the correlation coefficients for the aggregate properties and concrete strength were calculated. Correlation coefficients were considered to be significant at probability levels of 5 percent and less.

*Water absorption*—No significant correlation was found between the flexural or compressive strength of the three concrete mixes at either 7, 28, or 91 days, and the capacity of the aggregate to absorb water. The differences in the water-absorptive capacities of the aggregates used in this investigation were small and insufficient to cause significant variations in the effective water-cement ratios and, therefore, in the strength of the concrete. This does not rule out the possibility that highly porous aggregates, when used in a dry condition, may have a significant effect on the effective water-cement ratio and consequently on the strength of concrete.

*Aggregate strength*—As mentioned above, the crushing, flexural, and tensile strengths of the aggregates were closely related to their "crushing value." No significant relationship could, however, be established between the strength of the aggregates, as indicated by the crushing value test, and the flexural and compressive strengths of concrete.

Except in one instance (Aggregate K in Mix I at 91 days) the flexural and tensile strengths of the aggregates always exceeded the flexural strength of the concrete. In the one exception, the difference was small and could be due to random experimental errors. The crushing strengths of the aggregates were also all considerably higher than the compressive strength of the concrete. The possibility should not, however, be excluded that aggregates having much lower strength than those used in this investigation may affect the strength of concrete. It should also be mentioned that the conditions

of test for the flexural and compressive strengths of the aggregates were not the same as for the concrete. Effects arising from the different conditions of test are, however, not expected to be large.

As has been discussed elsewhere,[22] the flexural strength of concrete is generally lower than the flexural strength of the corresponding mortar. This suggests that an upper limit to the flexural strength of concrete is set by the strength of the mortar and that the presence of coarse aggregate generally reduces the flexural strength of the concrete to below that of the mortar. On the other hand, the compressive strength of concrete is usually greater than the compressive strength of its comparable mortar, which indicates that the mechanical interlocking of the coarse aggregate contributes to the ultimate strength of concrete when subjected to compressive loads.

*Shape*—As the author has shown,[15] there is a relationship between the flakiness and elongation of the aggregates used in this investigation, the correlation coefficient being +0.92.

Correlation coefficients for flakiness and concrete strength were calculated and it was found that in the case of flexural strength the coefficients were significant for Mix I at 28 days and Mix II at 28 and 91 days. None of the coefficients for compressive strength were significant.

The angularity number is also a measure of shape. For flexural strength, significant correlation with angularity was found for Mix I at 28 days and Mix II at 91 days, while for compressive strength, significant correlation was found for Mix I at 7 and 28 days and Mix III at 7 days.

These results suggest that the shape of the aggregate may have an effect on both the flexural and compressive strength of concrete.

*Surface texture*—Significant correlation was found between the surface texture of the aggregates and the flexural and compressive strength of Mix I concrete at 28 and 91 days. Although Aggregate K (Penryn granite) had the roughest surface texture, the flexural and compressive strengths of concrete made with this aggregate were comparatively low. When omitting this aggregate, the correlation coefficients for flexural strength were significant for all concrete mixes at all ages; in the case of compressive strength, five of the nine coefficients were significant.

There are therefore clear indications that the surface texture of the aggregate has an effect on both the flexural and compressive strength of concrete. As shown elsewhere, the shape and modulus of elasticity of the aggregate also have a marked influence on the strength of concrete and in the case of Aggregate K it appears that the influence of these two latter factors tends to mask the effect of surface texture.

*Dynamic modulus of elasticity*—The correlation coefficients for the dynamic modulus of elasticity of the aggregate and the flexural strength of the concrete were, with the exception of Mix I after 91 days, significant for all mixes at all ages. For compressive strength, none of the coefficients were significant; when omitting Aggregate K, which has a particularly low dynamic modulus, two of the coefficients, i.e., for Mix I at 28 and 91 days, were significant.

The indications are, therefore, that the modulus of elasticity of the aggregate has an effect on the strength of concrete.

*Poisson's ratio*—None of the correlation coefficients for Poisson's ratio of the aggregates and the flexural and compressive strengths of concrete were significant. When omitting the results for Aggregate K, however, seven of the nine correlation coefficients for flexural strength and one for compressive strength were found to be significant, which indicates a possibility that here again the other properties of Aggregate K may tend to mask the effects of the Poisson's ratio of this aggregate on the strength of the concrete. However, as described below, this difficulty may be resolved by a multiple regression analysis[23] of the results, which indicates that the Poisson's ratio of the aggregate had no effect on the strength of the concrete.

*Ultrasonic pulse velocity*—There was evidence of a relationship between the velocity of propagation of an ultrasonic pulse through the aggregate and the flexural, but not the compressive, strength of concrete.

## STATISTICAL REGRESSION ANALYSIS

Although correlation coefficients may serve to indicate whether a relationship exists between one particular property of coarse aggregates and the strength of concrete, it must be realized that the significance of the correlation is based on the premise that other properties of the aggregates do not affect the strength of concrete. This is not necessarily true, and a statistical analysis, which takes into consideration all the aggregate properties which may affect concrete strength, was used. The method employed was a multiple regression analysis as described by Brownlee.[23]

The correlation coefficients indicated that the strength of concrete may be related to the following properties of the aggregates: shape, surface texture, modulus of elasticity, Poisson's ratio, and pulse velocity. It has also been shown[15] that there is a relationship between the flakiness and the angularity of the aggregates used. A regression analysis revealed further that, when flakiness and angularity were considered together, the total variance accounted for in the strength of the concrete was only slightly greater than when angularity only was considered. For these reasons, the angularity number was used in the subsequent regression analysis to represent the shape properties of the aggregate. Ultrasonic pulse velocity was not included in the regression analysis because it is a function of dynamic modulus and Poisson's ratio, which are also the more fundamental properties. Therefore, in the first multiple regression analysis, the independent variables were: angularity, surface texture, elasticity, and Poisson's ratio. The dependent variables were, respectively, the flexural and the compressive strength of the concrete.

Neither for flexural nor compressive strength were any of the regression coefficients for Poisson's ratio significant. Therefore, there does not appear to be a relationship between the Poisson's ratio of the aggregates used in this investigation and the strength of the concrete made with them.

**TABLE 9—REGRESSION COEFFICIENTS FOR AGGREGATE PROPERTIES AND THE STRENGTH OF CONCRETE**

| Mix | Age, days | Average flexural strength, psi | Average compressive strength, psi | Regression coefficients | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Flexural strength | | | Compressive strength | | |
| | | | | Angularity | Surface texture | Elasticity | Angularity | Surface texture | Elasticity |
| I | 7 | 754 | 8340 | 5.34 | 4.26 | 20.59 | 74.43 | 10.15 | 54.14 |
| | 28 | 893 | 10450 | 16.92 | 6.88 | 28.76 | 82.28 | 74.20 | 90.82 |
| | 91 | 925 | 11260 | 10.82 | 10.05 | 16.71 | 57.99 | 77.34 | 129.32 |
| II | 7 | 508 | 4250 | 5.67 | 1.70 | 12.16 | 30.47 | −9.79 | 41.02 |
| | 28 | 665 | 6390 | 12.15 | 2.07 | 24.13 | −1.44 | 20.39 | 77.58 |
| | 91 | 717 | 7520 | 16.40 | 4.61 | 27.37 | 35.59 | 20.12 | 77.20 |
| III | 7 | 315 | 2200 | 3.75 | 0.28 | 6.21 | 30.54 | 3.35 | 24.29 |
| | 28 | 436 | 3410 | 4.82 | 2.65 | 14.06 | 10.30 | 10.80 | 35.93 |
| | 91 | 499 | 4240 | 6.38 | 1.96 | 13.03 | −4.56 | 20.66 | 44.27 |

The analysis was repeated, omitting Poisson's ratio and the regression coefficients obtained for flexural strength and compressive strength are given in Table 9.

### Shape

Three of the regression coefficients for angularity and flexural strength, and two for compressive strength, were statistically significant. All the coefficients differed significantly from each other. It therefore seems likely that concrete strength depends on the angularity of the aggregate, although the investigation was not sensitive enough to establish this in all cases.

By grouping the coefficients according to either the mix or the age of test, and taking the average, which will be more reliable than any individual value, a tendency is observed for the higher coefficients to be associated with conditions leading to higher strength. It is thus concluded that the effect of particle shape becomes more important at the higher strength levels.

### Surface texture

One of the regression coefficients for surface texture and flexural strength, and also one for compressive strength, were significant. All the coefficients also differed significantly from one another. As in the case of angularity, it therefore seems likely that the surface texture of the aggregate has an effect on both the flexural and compressive strength of concrete. The rougher the surface texture the greater the strength. As shown in Fig. 3(A) and 4(A), there is also a tendency for the magnitude of surface texture effects to increase as strength increases.

### Dynamic modulus of elasticity

Eight of the nine regression coefficients for dynamic modulus and flexural strength were significant. The coefficients were all positive and differed significantly from each other. As shown in Fig. 3(B) dynamic modulus effects also tend to be greater as the flexural strength increases.

None of the regression coefficients for dynamic modulus and the compressive strength of the concrete were statistically significant. An analysis of variance, however, showed that when the effects of angularity, surface texture,

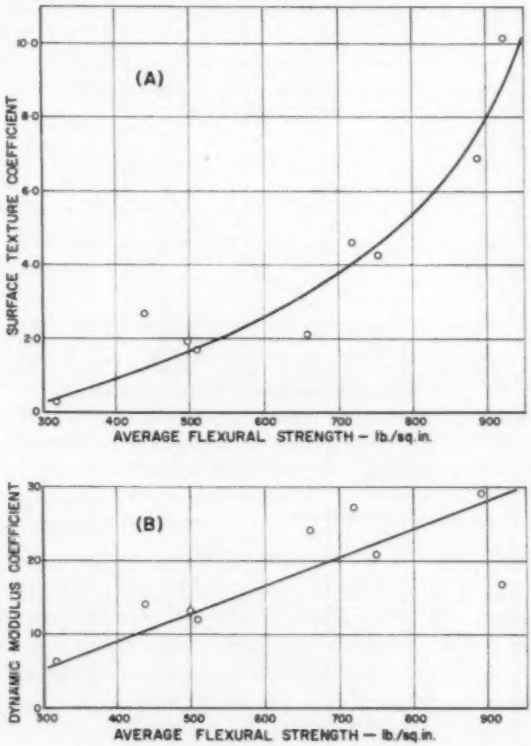


Fig. 3—Regression coefficients for aggregate properties and the flexural strength of concrete

*and* dynamic modulus were considered, 23 percent more of the variation in compressive strength was accounted for than when the dynamic modulus was not included. This, together with the fact that the coefficients were all positive and differed significantly from each other, suggests that the modulus of elasticity of the aggregate also has an effect on compressive strength. The indications are, however, not as clear as for flexural strength. As shown in Fig. 4(B), the effect increases with the compressive strength of the concrete.

## RELATIVE EFFECTS OF SHAPE, SURFACE TEXTURE, AND MODULUS OF ELASTICITY OF THE AGGREGATE

A statistical analysis of variance was carried out to ascertain to what extent the variation in concrete strength could be attributed to variations in shape, surface texture, and modulus of elasticity of the coarse aggregates.

In the case of flexural strength it was found that, on the average, for all concrete mixes at all ages, 84 percent of the strength variation was due to the shape, surface texture, and modulus of elasticity of the aggregate. The remaining 16 percent could have been due to random errors in testing the aggregates and the concrete. To confirm this conclusion, the regression equa-

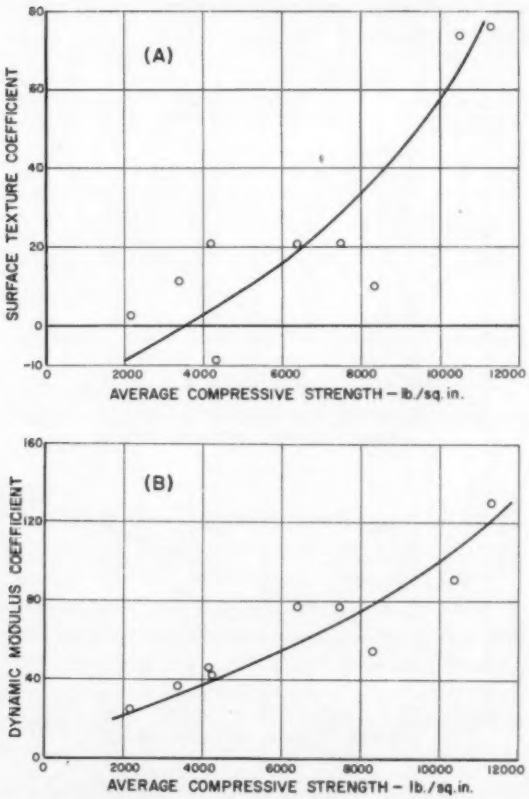


**Fig. 4—Regression coefficients for aggregate properties and the compressive strength of concrete**

tions for flexural strength and the properties of the aggregates, were calculated for each concrete mix at each age. The equation for the average flexural strength of all mixes at all ages, obtained by averaging the regression coefficients, is

$$y = 18.1e + 9.1a + 3.8s + 375 \quad \ldots\ldots(1)$$

where

$y$ = flexural strength of concrete (psi)
$e$ = dynamic modulus of aggregate ($\times 10^{-6}$ psi)
$a$ = angularity number of aggregate
$s$ = roughness factor (surface texture) of aggregate

The average flexural strength of the concrete for each aggregate was calculated from Eq. (1). The comparison of these calculated average strengths with the average observed strengths is shown in Fig. 5. It was shown statistically that the small differences between calculated and observed strengths may be accounted for by random errors in testing.

For the aggregates used in this investigation, it was therefore clear that, allowing for chance variations in testing, the variations in the flexural strength

of concrete were wholly due to variations in shape, surface texture, and modulus of elasticity of the aggregates.

In regard to compressive strength, the analysis of variance indicated that, for the strongest concretes, i.e., Mix I at 7, 28, and 91 days, 70 percent of the variation in strength may be attributed to variations in the shape, surface texture, and modulus of elasticity of the aggregate. For concrete of lowest strength, i.e., Mix III at 7, 28, and 91 days, 35 percent of the strength variation was accounted for. On the average, i.e., for all mixes at all ages, 52 percent of the variation was due to variations in the aggregate properties mentioned. It was established that, on the average, random experimental errors could account for the variations in strength which were not due to the above properties of the aggregate. It is nevertheless apparent that at lower strength levels the variations due to random experimental errors tended to mask variations caused by changes in properties of the aggregates.

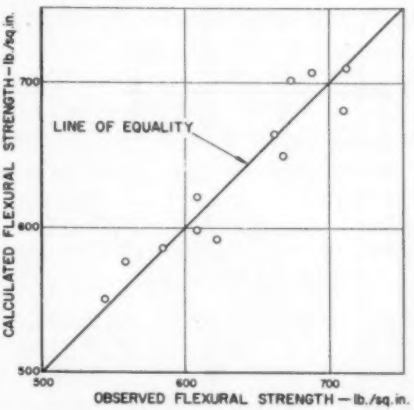


**Fig. 5—Average observed flexural strength of concrete made with different aggregates, compared with the average strength calculated from the regression equation**

The regression equation relating aggregate properties and the average compressive strength of the concrete for all mixes at all ages was found to be:

$$y = 64e + 35a + 25s + 5400 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where y = compressive strength of concrete (psi).

The average compressive strength for each aggregate was calculated from Eq. (2). These calculated strengths are compared with the average observed strengths in Fig. 6. As mentioned above, the differences between observed and calculated strengths could have been due to random experimental errors. It is therefore apparent, particularly at the higher strength levels, that, after allowing for chance variations in testing, variations in the compressive strength of the concrete were entirely due to variations in the shape, surface texture, and modulus of elasticity of the aggregates used in this investigation.

**TABLE 10—AVERAGE RELATIVE IMPORTANCE OF THE AGGREGATE PROPERTIES AFFECTING THE STRENGTH OF CONCRETE**

| Concrete property | Relative effect, percent, of the following aggregate properties: | | |
|---|---|---|---|
| | Shape | Surface texture | Modulus of elasticity |
| Flexural strength | 31 | 26 | 43 |
| Compressive strength | 22 | 44 | 34 |

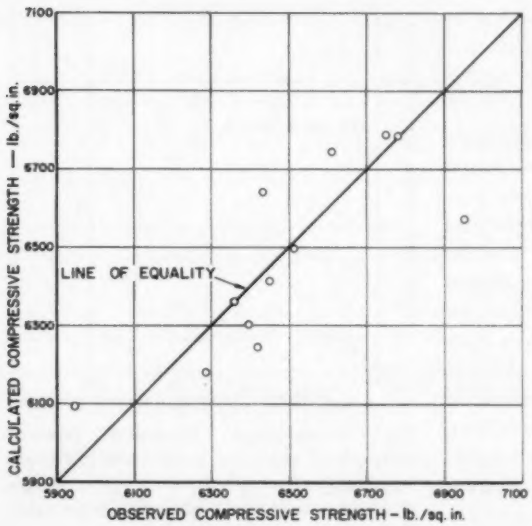


**Fig. 6—Average observed compressive strength of concrete made with different aggregates, compared with the average strength calculated from the regression equation**

It must be emphasized that Eq. (1) and (2) should not be considered as representing precise relationships between concrete strength and aggregate characteristics. They are the equations which best fit the data obtained in this investigation and show the general way in which concrete strength was affected by the properties of the aggregates used.

The relative importance of the shape, surface texture, and elasticity of the aggregates, with respect to concrete strength, was determined by expressing the variance due to each of the aggregate properties as a percentage of the total variance accounted for by all three characteristics of the aggregate. The results for the average of all concrete mixes are given in Table 10.

Although, on the average, the elasticity of the aggregate had the most important effect on flexural strength, for the concrete having the highest flexural strength (Mix I at 91 days), the surface texture had the greatest effect, accounting for 51 percent of the total variance. The shape of the aggregate was, on the average, the second most important factor.

In regard to compressive strength, surface texture was on the average the most important factor followed by elasticity and shape.

## CONCLUSIONS

1. Different coarse aggregates have a marked effect on the strength of concrete of the same mix proportions. Depending on the aggregate, differences of 40 percent in flexural strength and 29 percent in compressive strength were obtained.

2. The aggregate properties affecting the flexural and compressive strength of concrete were shape, surface texture, and modulus of elasticity. These effects became more important the greater the strength of the concrete.

The elasticity of the aggregate was, in general, the most important single factor affecting flexural strength, although for concrete with the greatest strength surface texture had the predominating effect. Surface texture was, in general, the most important factor affecting compressive strength.

A possible explanation as to why rougher surface texture of the aggregate increases concrete strength may be that it results in a greater adhesive force between the cement matrix and the aggregate. The effect of greater angularity may be due to the fact that the surface area of an aggregate particle becomes larger the more its shape deviates from a spherical form. The total adhesive force between the cement matrix and an angular aggregate may, therefore, be greater than for a rounded aggregate. The deformation of the aggregate in concrete under a given load will no doubt also influence the strength of the concrete; the greater the deformation, i.e., the lower the elastic modulus, the lower the strength.

3. The aggregates used were generally stronger than the concrete and no relationship between the strength of the coarse aggregates and the strength of concrete was established. This finding should not be taken to mean that aggregates of low strength will not affect concrete strength.

The upper limit to the flexural strength of the concrete was set by the strength of the mortar. Presence of coarse aggregate generally reduced the flexural strength of the concrete to below that of the mortar. On the other hand, concrete compressive strength was generally greater than that of the mortar; the presence of coarse aggregate therefore contributed to the ultimate compressive strength of the concrete.

4. Differences in the water-absorptive capacities of the aggregates used in this investigation were too small to have a significant effect on the strength of concrete. It may be that more porous aggregates, when used dry, will affect concrete strength.

## ACKNOWLEDGMENTS

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Title No. 55-73

Concrete walls 4 to 6 ft thick, constructed inside 50-ft diameter steel chambers at the Shippingport, Pa., atomic power plant, serve as shielding against radioactivity. Paper describes unusual construction practices occasioned by limited access to chamber interiors. Concrete was a standard density, 3000-psi mix made with Type I S cement.

# Internal Shielding Construction at Shippingport Nuclear Power Plant

By L. EARL TABLER, Jr.

AMONG MANY CONSTRUCTION PROBLEMS encountered during the erection of the nation's first full-scale nuclear power plant at Shippingport, Pa., building the concrete internal shielding walls presented a unique challenge. The reactor plant container, which houses the nuclear portion of the power station, consists of four steel pressure chambers interconnected by 8 ft and 12 ft diameter ducts (Fig. 1). The reactor vessel is located in a 38 ft diameter steel sphere. On each side of the sphere is a 50 ft diameter, 97 ft long cylinder, each of which contains two heat exchangers (boilers) and their associated equipment. Between the boiler chambers and the turbine-generator portion of the plant is the auxiliary chamber which is a steel cylinder 50 ft in diameter and 147 ft long. In the auxiliary chamber are the primary coolant pressurizer, flash tank, blow-off tank, and other auxiliary equipment.

The chambers are divided into separate compartments by concrete walls (Fig. 1) from 2 to 6 ft thick, which serve as shielding against radioactivity. The wall thickness depends on the amount of access required into the various compartments for operation or maintenance. The shielding in the boiler chambers is designed so that any one boiler can be shut down for observation or repairs while the remaining three are still in operation. Thicker shielding at opposite ends of the boiler chambers, hexagonal in shape, houses the demineralizers for the primary coolant water. The shielding in the boiler chambers is supported directly by each steel shell.

The shielding in the auxiliary chamber is not required to be as thick as that in the boiler chambers, with the exception of the walls in front of each interconnecting duct. The walls in this chamber provide a separate compart-

ment for the pressurizer and for the flash and blow-off tanks, and are supported on structural steel members which in turn are supported by the steel shell (Fig. 2).

## SHIELDING CONCRETE

Heavy concrete was not used for these internal radiation shielding walls. The standard 3000-psi mix contained 5½ sacks of Type I S cement per cu yd. The coarse aggregate was river gravel, 1½ in. maximum size. Slump was specified at 4 in. Some difficulty was encountered in pumping this mix when the weather was quite warm. To overcome this, an extra sack of cement was added per cu yd so that the slump could be increased to 6 and 7 in. without impairing the strength. Despite the high slump mix, no shrinkage problems were encountered.

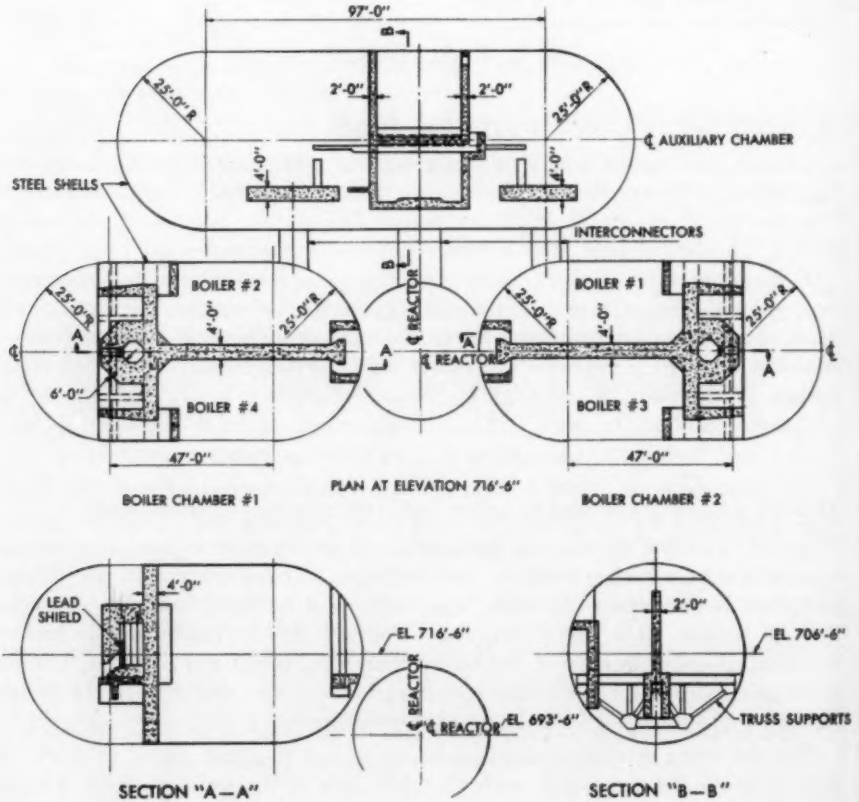


Fig. 1—Plan and elevation of steel chambers for boilers, reactor, and auxiliary units with general details of internal shielding

**L. EARL TABLER, JR., a civil engineering graduate of the University of Iowa, is in charge of the construction planning and scheduling department of the Machinery Division, Dravo Corp., Pittsburgh. Following several years of experience in proportioning and control of concrete mixes and placement on conventional steam plant construction, Mr. Tabler served as job engineer 2 years at the Shippingport atomic power plant. There he was responsible for all field engineering relating to construction of the nuclear portion of the plant.**

## CONSTRUCTION PROBLEMS

### Access and handling

Limited access to the containers was responsible for most of the problems encountered in placing the 2750 cu yd of concrete required for the internal shielding walls. In the top of each boiler chamber there are two 10 ft diameter and one 6 ft diameter hatchways. In the top of the larger auxiliary chamber there are only two 10 ft diameter hatchways.

Added to the limited access situation was the fact that all materials had to be handled manually once inside the chambers. For ease of handling, it was decided to use 2 x 4-ft, 2 x 6-ft, and 2 x 8-ft prefabricated panel forms with a plywood face. The portions of the forms that had to be cut on a radius were prefabricated on the job.

Most of the reinforcement (Fig. 3) in the heavier walls consisted of #11 bars spaced at 12 in. horizontally and vertically in each face. This presented another handling problem, for long #11 bars are quite heavy. Because of limited storage space inside the container, each item of forms or reinforcing had to be lowered through the hatchways in such a sequence that it could be installed almost immediately. This required close coordination and co-operation among the various craftsmen involved.

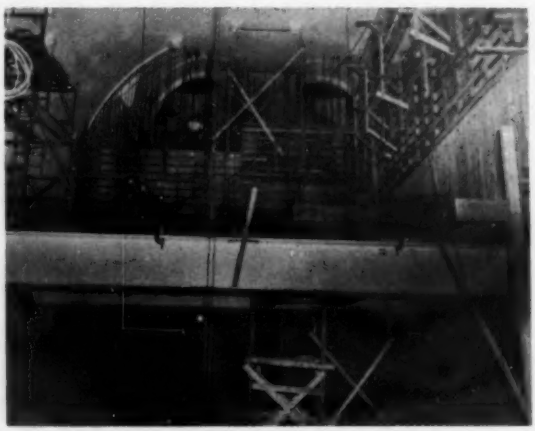

Fig. 2—Structural steel members spanning the lower section of the steel shell supported shielding concrete walls in auxiliary chamber

Fig. 3—Forms and reinforcement being erected for first lift of concrete inside boiler chamber. Forms were prefabricated in panels sized for easy handling inside the chambers

The boiler chambers were designed to support the shielding walls after the concrete was set, but while the concrete was being placed, auxiliary supports were needed beneath the steel shells. Heavy 12 x 12-in. timber shoring (Fig. 4) between the 7-ft thick foundation slab of the external concrete enclosure and the outside of the steel shells was used for this purpose.

**Placing the concrete**

The walls in the boiler chambers were placed in four lifts, three of 15½ ft and one of 3½ ft. In the auxiliary chamber, the height and number of lifts varied with the walls involved.

Construction of the external portion of the power plant* continued while the concrete was being placed inside the container. To minimize all possible interferences with this construction, it was decided to use a positive displacement concrete pump to get the concrete to the container hatchways. The pump selected was capable of pumping 20 cu yd of 3000-psi, 6-in. slump concrete per hr through approximately 100 ft of 6-in. steel pipe (about 80 ft horizontally and 20 ft vertically). The concrete was pumped into a hopper located just inside the particular hatchway involved, thence dropped into the forms via an 8-in. flexible rubber hose.

*Concrete 5 ft thick shields the outside of these four steel containers. A brief article on construction of the exterior block houses appeared in the ACI JOURNAL, News Letter, May 1957, pp. 25 and 27.—EDITOR

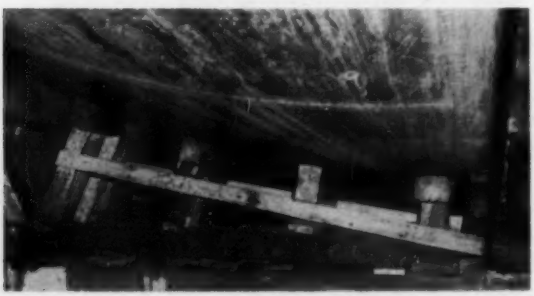

Fig. 4—Shoring of 12 x 12-in. timbers between steel shell and heavy foundation slab supported weight of the internal shielding walls until concrete had hardened

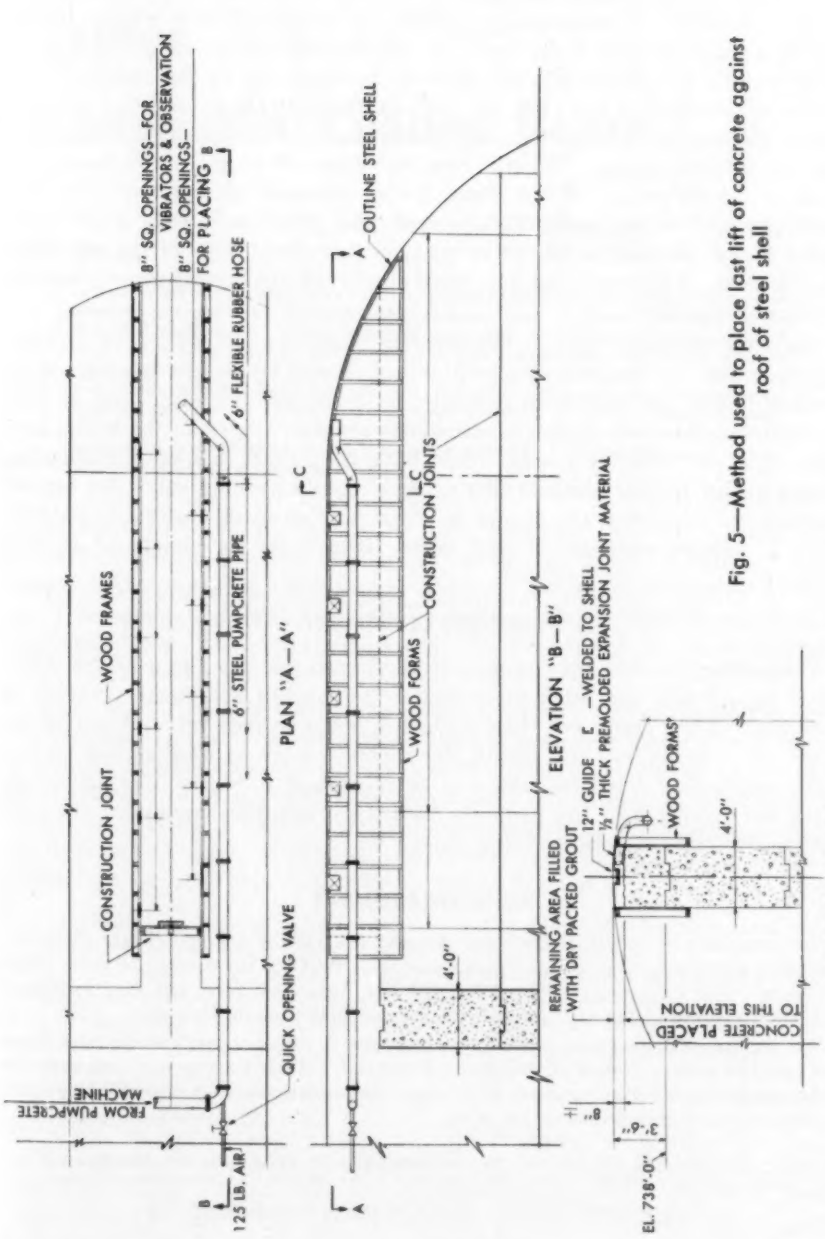


Fig. 5—Method used to place last lift of concrete against roof of steel shell

This method of placing was used for all lifts except the last one inside the boiler chambers. It was necessary that the last lift of concrete be placed tightly against the roof of the shell. To accomplish this, concrete was pumped directly into the forms through 8 x 8-in openings cut in the sides of the forms at the extreme top (Fig. 5). A 6-in. flexible rubber hose was adapted to the discharge end of the concrete pump steel pipe line to facilitate getting the line into the forms. When it became impossible to place any more concrete in a given section of the forms, it was necessary to shut down the concrete pump, remove a section of the pipe, and place the hose in a new position. It was possible to fill the forms to within about 8 in. of the top using this method. The remaining 8 in. were filled with dry packed grout after the forms were removed.

Since the discharge pipe of the concrete pump always had to be full during operation, and remain full until it was cleaned by external means, some method had to be devised to partially clean the last horizontal run of pipe to facilitate the removal of the individual sections. To clean the entire line, starting at the concrete pump, would have been much too time consuming. Therefore, an auxiliary air line (Fig. 5) with a quick opening valve was tapped into the end of this last run of pipe to "blow out" as much concrete as possible when it became essential to shut down temporarily for change of placing position.

## SURFACE TREATMENT

Preparatory to the application of a vinyl base paint to the concrete walls, all of the air pits and other voids had to be patched and rough protrusions removed. A 1:2 grout was then rubbed into the surface with a hand stone. After the rubbing was completed, the concrete surfaces were etched with a weak solution of muriatic acid and then washed down with clean water. With the exception of the painting itself, this operation completed the construction of the concrete shielding walls.

## ACKNOWLEDGMENT

The steam electric portion of the plant, designed and built by Duquesne Light Co. in cooperation with the Atomic Energy Commission, is on the Ohio River about 25 miles below Pittsburgh, near Shippingport, Pa. Burns and Roe, Inc., New York, and their associates, J. Rich Steers, and Hatzel and Buehler, were agent-constructors for Duquesne.

Westinghouse Electric Corp., under contract with the Atomic Energy Commission, designed and built the nuclear portion of the plant. Stone and Webster Engineering Corp. acted as architect-engineer for Westinghouse, and Dravo Corp., under contract with Westinghouse, constructed the nuclear portion of the plant.

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Title No. 55-74

# Further Tests of Dynamically Loaded Beams

By F. T. Mavis and J. J. Stewart

**Twenty-seven beams, identical except for amount and grade of reinforcement, were tested to destruction by spring-actuated impulse loads applied at the third-points of an 8-ft simple span. Loads, reactions, bar strains, and deflections were recorded simultaneously throughout each test by a motion picture camera at speeds of 2000 pictures per sec. Frame-by-frame analysis of the pictures showed: (1) all beams were destroyed when deflection reached about 6 in. (or 6 percent of span length) at midspan; (2) at the onset of destructive failure steel strain was less than 4 percent; (3) no reinforcing bar fractured in any test; and (4) beams with hard grade bars consistently outperformed beams with intermediate or structural grade bars under otherwise identical conditions.**

STUDIES RECENTLY PUBLISHED[1,2] reported dynamic loading tests of reinforced concrete beams identical except for grade of reinforcement, loaded at the middle of a 6-ft simple span by a spring-loaded testing machine which released a single destructive impulse between two beams cradled pairwise. Those beams were reinforced with 1.6 percent of either structural grade, intermediate grade, or hard grade steel.

There was evident need for more information on the behavior of dynamically loaded beams with different percentages of steel as well as different grades. Moreover, experiments wherein loads were applied dynamically at the third points instead of at midspan were called for. It was soon obvious that the apparatus previously used to test beams pairwise under a single impulse was inadequate. A fresh start was necessary. Accordingly, the problem of supplementing and extending earlier studies involved:

1. Building a new spring-loaded testing machine with more than four times the capacity of the one used in earlier experiments.
2. Devising a trigger mechanism that would be adequate for these increased loads and be positively controllable within a time tolerance of about 0.2 sec.
3. Recording simultaneously (a) four oscilloscope traces on the emulsion side of film at 2000 pictures per sec in a high speed motion picture camera and (b) photographs of a test in progress.
4. Analyzing data, frame by frame, from a motion picture record of each beam individually loaded to destruction.

This paper covers these points, and summarizes the data on all tests at 13.3 kips spring-load to show the relative performance of like-sized beams with one or two reinforcing bars of either structural grade, intermediate grade, or hard grade steel.

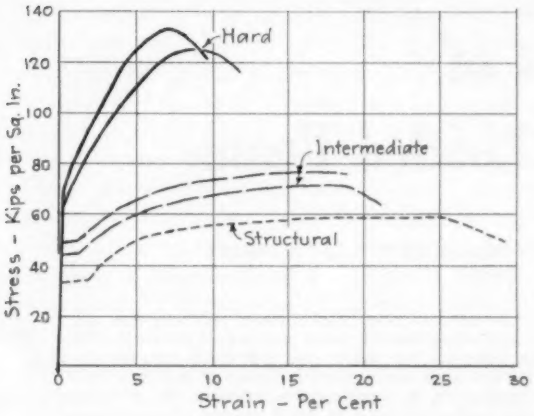


**Fig. 1—Stress-strain curves for three grades of steel used in test beams**

## SPECIMENS AND EQUIPMENT

All beams were 11.0 in. wide, 5.8 in. deep, and 8 ft 6 in. long. One or two #5 bars placed 4.7 in. from the compression face provided 0.6 or 1.2 percent of steel.

Bars were selected from six lots of steel including one structural grade, two intermediate grades, and three hard grades. Fig. 1 shows the stress-strain curves for steel specimens.

Beams were cast in gang forms 12 at a time. They were kept wet under burlap or heavy paper 7 days and then kept moist for 21 days. Standard 6 x 12-in. cylinders tested at 28 days for each of three groups of beams showed strengths of 5600, 5300, and 6300 psi, respectively.

### Testing machine

Fig. 2 is a schematic diagram of the testing machine. The test beam (1) was supported on two 9-in. lengths of 6-in. pipe (2) which were welded to posts (3) supported on the base of the testing machine. The base was an 18-in. wide-flange beam (4) with web horizontal spanning an open hatchway.

Two helical springs (5) were 45 in. long (unloaded) and capable of exerting a combined force of 24 kips. Each spring had a modulus of 1.28 kips per in. These springs were housed in sections of 6-in. pipe bolted to the underside of the wide-flange base beam.

A needle beam (6) of two 6-in. channels 62 in. long was connected to the bottom of each of the loading springs (5) by two 1-in. diameter rods (7). These rods could be adjusted by nuts at the top to transmit a predetermined initial load from the spring to the test beam.

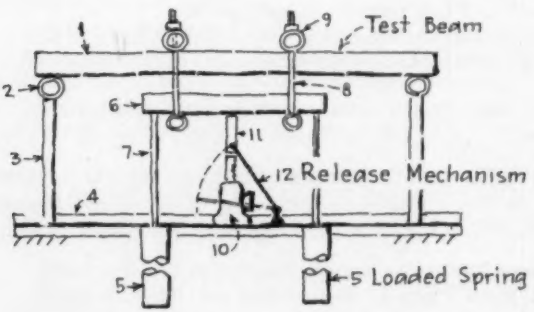


**Fig. 2—Schematic diagram of the loading apparatus**

Two load yokes which transmitted load from the needle beam (6) to the test beam (1) consisted of two threaded rods each, front and back of the beam (8), and load rings (9) made of 9-in. lengths of 6-in. diameter double extra strong pipe with a load transferring web.

The trigger mechanism consisted of a 15-ton quick release track jack (10) as used in railroad maintenance and a two-bar linkage. The linkage was made up of a slug or strut (11) hinged near its bottom to another link (12) which pivoted about an axis at its lower end. This axis was fixed relative to the base of the testing machine.

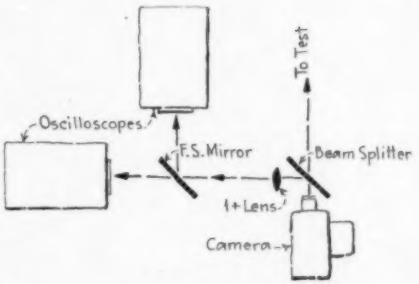


Fig. 3—Optical system

In principle the operation of the testing machine was simple. The springs were loaded by hydraulic jacks which raised the needle beam (6) relative to the testing machine base (4). The trigger jack (10) was then raised to its highest position and the slug (11) was inserted, closing the gap between the extended jack and the needle beam. Slugs of different length were available.

When the quick release jack was triggered the load was suddenly transferred from the jack to the system which included the load yokes (8), load rings (9), test beam (1), reaction rings (2), and struts (3) supported on the base beam (4).

This testing machine, used repeatedly in a loading range from 10 to 20 kips, proved adequate and reliable. With additional springs and details for which provision has been made, it will have a maximum capacity of 48 kips.

## Instrumentation

Electric resistance strain gages (SR-4, type C) were attached to the load rings, reaction rings, and reinforcing bars, and four strain signals were recorded on two dual-beam oscilloscopes.

A motion picture camera operating at approximately 2000 frames per sec recorded data from the oscilloscopes and simultaneously photographed the beam during a test. The images of the oscilloscope faces were superimposed on the same optical plane by means of a front surface mirror. These images were then brought into focus with the image of the test beam by means of a supplemental lens and a dichroic mirror as shown in Fig. 3. On a black screen behind the loading apparatus and test beam, outboard in the field of view, two bright targets

**F. T. MAVIS, dean of engineering at the University of Maryland, College Park, formerly headed the Department of Civil Engineering at Carnegie Institute of Technology where these dynamic loading tests were conducted. Dean Mavis, an ACI member since 1947, was corecipient of the 1958 Wason Medal for research, presented in recognition of work reported in "Destructive Impulse Loading of Reinforced Concrete Beams" (ACI JOURNAL, September 1957).**

**J. J. STEWART is assistant professor of civil engineering at Carnegie Institute of Technology, Pittsburgh. His study of these dynamic loading tests was part of his work for the PhD degree which he received at Carnegie Tech in 1957. Dr. Stewart, who joined the Institute in 1951, was a structural designer for Richardson, Gordon and Associates, Pittsburgh, prior to assuming his present academic duties.**

marked the initial positions of the oscilloscope dots. In this way it was possible to photograph on the emulsion side of the film both the "event" on the loading stage and the "record" of strain as picked up by four dots on the oscilloscopes. With the available equipment it would have been impossible otherwise to record oscilloscope traces through the antihalation backing of the film at speeds of 2000 pictures per sec.

## TEST RESULTS

Fig. 4 shows static load-deflection curves for representative beams similar to those tested dynamically. Each beam is coded with an alternating sequence of numbers and letters. *I* identifies intermediate grade of steel, and *R* indicates hard grade. *A* and *B* identify one-bar and two-bar beams, respectively.

Fig. 5 is an action photograph taken about 60 millisec after a load of 20 kips was applied to a beam reinforced with two hard grade bars. This load was the greatest applied in any of the tests. At the instant Fig. 5 was photographed, the concrete had exploded at midspan.

The horizontal white line across the load rings (Fig. 5) is a thin layer of dry sand which is falling freely from its initial position on the surface of the unloaded beam. Note the discontinuity of this "sand line" near the ends of the beam, (a) where the sand has just remade contact with the deflected beam, and (b) beyond that where the dry sand has rebounded upward from the dynamically loaded beam.

In Fig. 5, the slug (Point 11 in Fig. 2) has cleared its upper bearing on the needle beam by some 2 in. The trip-jack has lurched to the right about 6 in. under the impulse triggered by the load-release mechanism. Fig. 5 gives some impression of the violence of impulse loading.

Fig. 6 shows the same beam after the dynamic loading test of 20 kips. The concrete is crushed at midspan, and it is "sliced off" the sides of the beam throughout most of the middle third. Yet the reinforcing bars—in this

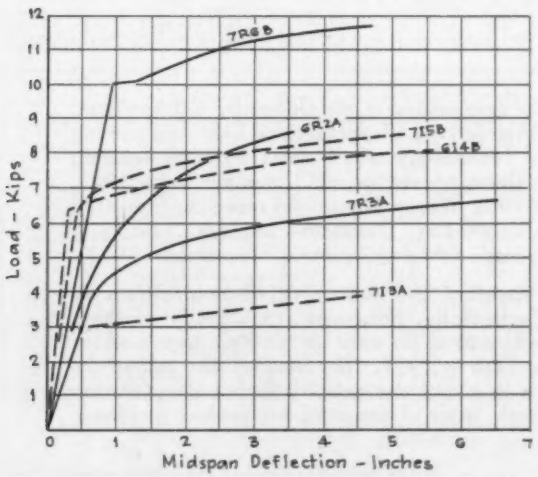


Fig. 4—Static load-deflection curves for representatative beams

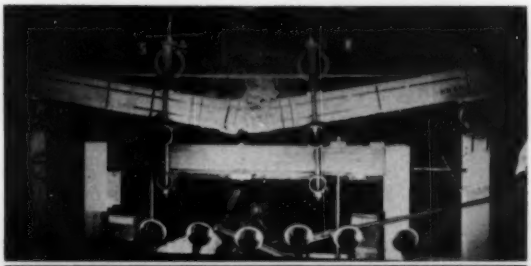

Fig. 5—Beams with two hard grade bars during 20-kip dynamic test. Horizontal white line is layer of dry sand spread on the beam before test

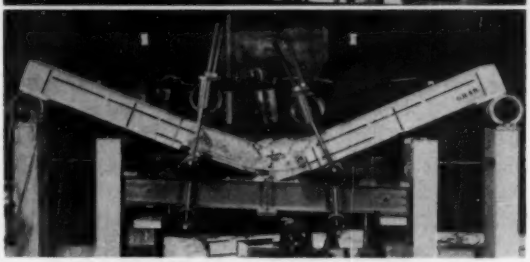

Fig. 6—Same beam as in Fig. 5 after 20-kip dynamic test

case hard grade steel—are merely strained and bent. There are no indications of impending steel failure.

Fig. 7 shows in more detail the midsection of another two-bar beam and a similar, typical failure.

### Vertical splitting of one-bar beams

Fig. 8 is a close-up plan view of a beam with one bar after a destructive dynamic test. The concrete was split vertically above the reinforcing bar throughout the middle third of the beam and about 4 in. beyond. Splits like this occurred in several tests of one-bar beams loaded dynamically, but never in tests of beams loaded statically. Just how they developed in dynamically loaded beams is not at present clear, but something akin to "whiplash" may be involved.

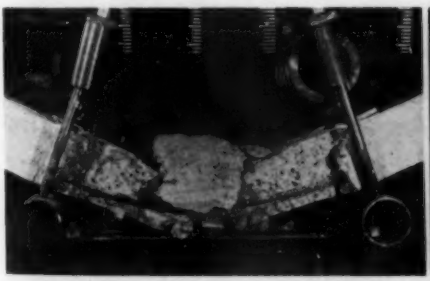

Fig. 7—Detail of failure of two-bar beam

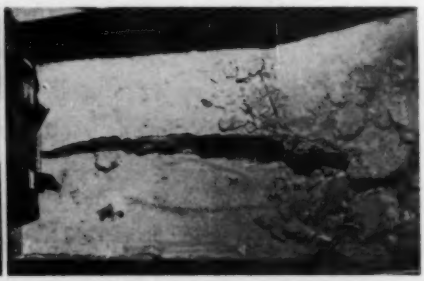

Fig. 8—Plan view of one-bar beam showing longitudinal split after dynamic test

The splits occurred directly above the bar, as if the bar had been struck a sharp hammer blow upward and transverse to the beam. Yet, unless the beam were moving toward the "hammer" the concrete on the top face of the beam (normally the compressive face) would fail in tension—not by crushing as it did in the beam shown in Fig. 8. Moreover, as is evident in Fig. 7 for a two-bar beam, the steel was stripped from the concrete in the characteristic after-appearance of a bond failure in the middle third of the beam where the static shear stress (and "theoretical" bond?) is zero. This opens another avenue of speculation. In these tests, hard grade bars yielded when the midspan deflection of beams was about 1 in. The corresponding vertical component of bar stress in the midsection of beam was about 1 kip. In a dynamic test the beams deflected at midspan at a rate of about 130 in. per sec. How these pieces of evidence—and others—fit together (with the stripping of concrete from the steel in the midsection only) may be clear later. At present it would seem worth noting practically, (a) that the convention of "balanced design" for static loading is shaken up a bit in designing and detailing logically for dynamic impulse loading, (b) that high strength concrete and high yield steel would seem to be a logical combination of materials for reinforced concrete structures intended to resist dynamic impulse loads, and (c) that "details" of reinforced concrete construction for dynamic loadings should be viewed from a firmer base than the conventions that have long been adequate for static loadings.

### Test data summarized

Table 1 summarizes all data collected in the dynamic tests. Beams were coded by number and letter, as explained in the table. Initial spring loads, residual midspan deflections after the test, and residual loads are tabulated.

TABLE 1—SUMMARY OF TEST RESULTS

| Beam No. | Maximum load, kips | Maximum midspan deflection, in. | Residual load, kips | Beam No. | Maximum load, kips | Maximum midspan deflection, in. | Residual load, kips |
|---|---|---|---|---|---|---|---|
| 1SA | 9.8 | 4.4 | 1.5 | 15RA | 13.3 | 5.0 | 4.5 |
| 2IA | 9.8 | 4.0 | 1.9 | 16IA | 13.3 | 7.2 | 0.6 |
| 3SA | 13.3 | * | 0.0 | 17IB | 13.3 | 4.0 | 7.0 |
| 4RA | 13.3 | 4.9 | 3.4 | 18RB | 13.3 | 2.1 | 10.0 |
| 5IA | 13.3 | 7.4 | 0.0 | 19IB | 16.0 | 24.0* | 0.0 |
| 6SB | 13.3 | 4.8 | 4.6 | 20RB | 16.0 | 19.0* | 0.0 |
| 7IA | 13.3 | 7.8 | 0.0 | 21RB | 16.0 | 3.0 | 10.6 |
| 8RA | 13.3 | 8.8 | 0.6 | 22SB | 17.5 | 10.5 | 0.0 |
| 9RA | 13.3 | 6.0 | 2.4 | 23IB | 17.5 | 10.0 | 0.0 |
| 10IB | 13.3 | 4.0 | 6.2 | 24RB | 17.5 | 19.8* | 0.0 |
| 11RB | 13.3 | 2.8 | 8.9 | 25RB | 17.5 | 21.5* | 0.0 |
| 12RA | 13.3 | 20.5* | 0.0 | 26RB | 17.5 | 4.2 | 10.2 |
| 13RA | 13.3 | 5.0 | † | 27RB | 20.0 | 17.5* | 0.0 |
| 14RA | 13.3 | 5.5 | 3.8 | | | | |

*Needle beam struck the wood blocking.
†Not recorded.
NOTE: S = structural grade reinforcement.
I = intermediate grade reinforcement.
R = hard grade reinforcement.
A indicates one-bar beams; B indicates two-bar beams.

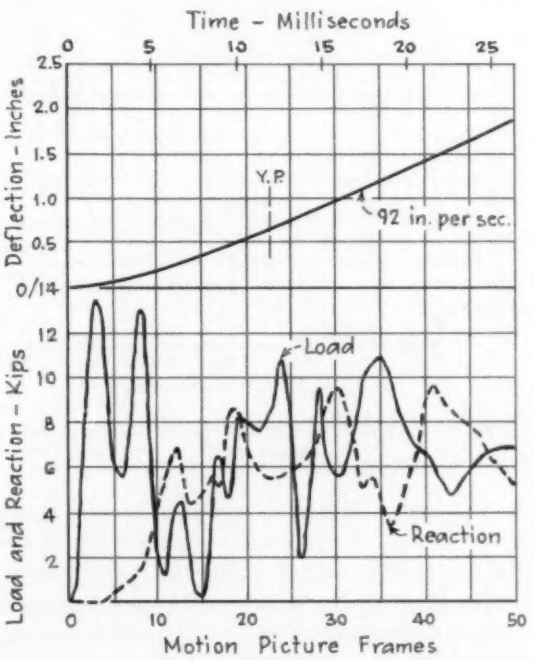


Fig. 9—Typical record of data from a dynamic test (Beam 4RA)

The first two tests were made at a spring load of 9.8 kips which was evidently less than even the one-bar beams could withstand. The next initial spring load explored, 13.3 kips, proved to be discriminating for beams with one bar or two bars, and for all grades of steel. Accordingly, 16 tests were made at this load. Nine two-bar beams were tested at initial spring loads ranging from 16.0 to 20.0 kips—one with structural grade bars, two with intermediate grade bars, and six with hard grade bars. Only two of the beams loaded in the 16–20-kip range were load-worthy after the dynamic test, and both were reinforced with hard grade bars. In both cases the residual load was more than 10 kips, or about 20 percent greater than the static ultimate load of otherwise similar beams reinforced with two intermediate grade bars.

Fig. 9 shows data obtained from a typical test of a one-bar beam reinforced with hard grade steel. The steel yielded about 13 millisec after the load was applied, and the beam deflected at the rate of 92 in. per sec soon thereafter.

By plotting bar strains and midspan deflection frame by frame from the motion pictures and observing abrupt changes in slope of resulting curves, the time and midspan deflection corresponding to nonlinear behavior (yield) were noted. The midspan deflection at which this nonlinearity was evident in this study averaged 1.1 in. for beams reinforced with hard grade bars, and 0.9 in. for beams with intermediate grade bars.

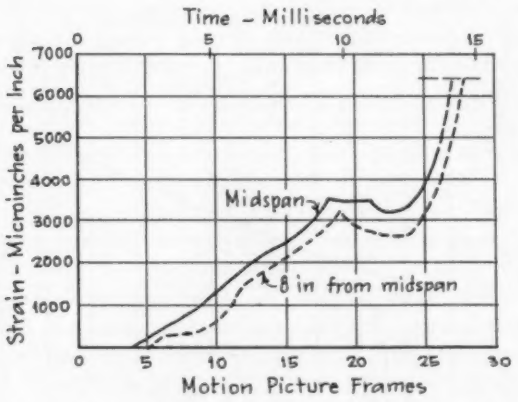


**Fig. 10—Typical bar strains during dynamic test (Beam 4RA)**

Ordinates in the lower part of Fig. 9 show load and reaction as indicated by the oscilloscopes. This tangled record is difficult to interpret in detail. However, it is apparent that the load on the beam, as indicated by strains in the load ring, reached its maximum about 2 millisec after the spring was released, and fluctuated violently as shown by the solid line. The reaction ring indicated no strain during the first 2 millisec of a "whiplash" phase. Thereafter, reaction fluctuated with time, but less violently than load. Eventually (beyond the limits of Fig. 9) the beam came to rest at a residual load (and reaction) of 3.4 kips and a residual midspan deflection of 4.9 in.

Fig. 10 shows bar strains for Beam 4RA (the same beam as in Fig. 9) plotted against time as the abscissa. The lines are cut off at the limiting strain recorded by the instruments. Flattened portions of the curves (at strains of about 3000 microin. per in.) may indicate adjustments that occur when bars slip or concrete cracks.

Fig. 11 shows all data obtained from tests at 13.3 kips initial spring load. Ordinates are residual load in kips after the test, and abscissas are corresponding residual deflections. Indicated by symbols 1-S are tests of beams with one structural grade bar; 1-I with one intermediate grade bar; 1-R with one hard grade bar. The symbols 2-S, 2-I, and 2-R denote corresponding two-bar beams. Observe that the tests plotted in this way fall into an orderly pattern—the greater the residual load, the less is the residual deflection (obviously) since the initial load is constant.

## CONCLUSIONS

The significant conclusions from these tests may be drawn from Fig. 11 as follows:

1. With just one exception, beams reinforced with one hard grade bar showed greater residual load capacity after dynamic test than beams reinforced with one structural grade or intermediate grade bar. In fact the beam with two structural grade bars behaved almost identically with the best of the beams with one hard grade bar.

2. Beams with two hard grade bars showed the highest residual load capacity after a dynamic test, followed in descending order by beams

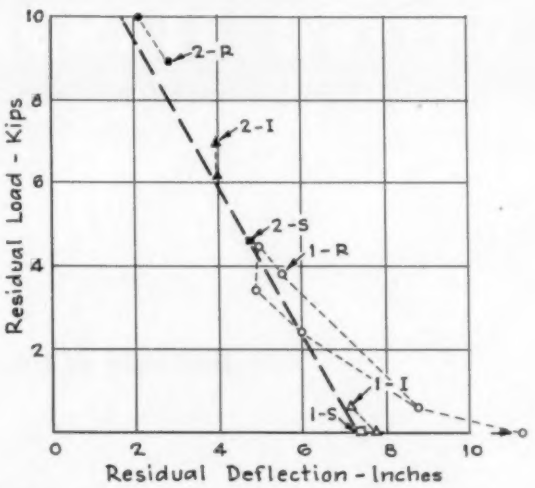


Fig. 11—Residual load-deflection data for tests at 13.3 kips maximum load

reinforced with two intermediate grade bars and two structural grade bars, respectively.

These conclusions are consistent with those drawn from earlier studies.[1,2] The mechanics of beam splitting under dynamic loading (which occurred in several tests as shown in Fig. 8) may deserve further comment and experimentation.

## ACKNOWLEDGMENTS

The experiments reported in this paper were conducted at Carnegie Institute of Technology in cooperation with the Rail Steel Bar Association. They are reported in greater detail in a thesis by J. J. Stewart submitted in partial fulfillment of the requirements of the degree Doctor of Philosophy. Graduate students, whose help is gratefully acknowledged, included John Cernica, Ramon Garcia, John A. Hribar, Paul H. Sanders, and John R. Verna.

* * * *

Unpublished reports of experiments on concrete beams subjected to destructive loads have come to the authors' attention since the studies reported in this paper were finished. The reports covered tests of beams larger than those described in this paper. Authors of those reports are R. H. Carlson and J. P. Murtha, and W. A. Shaw and J. R. Allgood.

## REFERENCES

1. Mavis, F. T., and Greaves, M. J., "Instrumentation and Method for Impulse Testing of Reinforced Concrete Beams," *ASTM Bulletin*, No. 226, Dec. 1957, pp. 43-47.

2. Mavis, F. T., and Greaves, M. J., "Destructive Impulse Loading of Reinforced Concrete Beams," ACI JOURNAL, V. 29, No. 3, Sept. 1957 (*Proceedings* V. 54), pp. 233-252.

Received by the Institute June 27, 1958. Title No. 55-74 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 11, May 1959 (Proceedings V. 55). Separate prints are available at 50 cents.

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**Discussion of this paper should reach ACI headquarters in triplicate by Aug. 1, 1959, for publication in the Part 2, December 1959 JOURNAL.**

# CONCRETE BRIEFS

## Notes from Field and Office

Bond in Flat Slabs . . . . . 1225
Problems and Practices . . . . . 1227

## Bond in Flat Slabs

By ELIHU GEER*

Chapter 10 of ACI 318-56, which deals with the design of flat slabs, makes no mention of bond stress. Formula (6) of the Code evidently does not apply to a flat slab (Fig. 1).

The writer feels that bond stress might be critical in a flat slab without drop panel and capital (in other words a "flat plate"). If it is critical, then the code should make some mention of bond. A similar bond condition exists in footings although it is less critical.

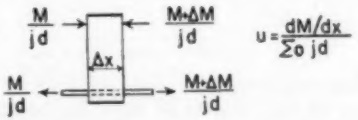


Fig. 1—Computation of bond stress formula

Timoshenko in *Theory of Plates and Shells* gives these differential equations for axially symmetric thin plates:

$$\frac{d}{dr}\left[\frac{1}{r}\frac{d}{dr}\left(r\frac{dw}{dr}\right)\right] = \frac{Q}{D}$$

$$M_r = -D\left[\frac{d^2w}{dr^2} + \frac{\nu}{r}\frac{dw}{dr}\right]$$

$$Q = -\frac{q\pi a^2}{2\pi r} + \frac{qr}{2}$$

$r$ = radial distance
$w$ = deflection
$Q$ = shear per unit width
$D$ = something like $EI$
$M_r$ = radial moment
$\nu$ = Poisson's ratio
$q$ = unit transverse load

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*Member American Concrete Institute, Professor and Chairman, Department of Civil Engineering, University of Detroit, Detroit, Mich.

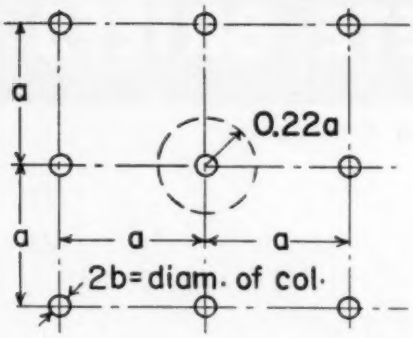


Fig. 2—Limit of radial moment in a flat plate floor

He also suggests the approximation that in a structural system such as a flat plate floor the radial moment is zero at a distance $0.22a$ from center of column (Fig. 2).

Using this approximation the ratio of rate-of-change-of-radial-moment to shear-at-face-of-column equals

$$\frac{dM_r/dr}{Q} = -\tfrac{1}{2}(1+\nu) + \frac{\dfrac{(1-\nu)}{0.1936}\left(\dfrac{b}{a}\right)^4 - \dfrac{(1+\nu)}{\pi}\cdot\ln\left(\dfrac{0.22a}{b}\right) - \dfrac{1}{\pi} + 0.0121\,(3+\nu)}{\left[\dfrac{1}{\pi} - \left(\dfrac{b}{a}\right)^2\right]\left[\dfrac{(1+\nu)}{(1-\nu)} + \left(\dfrac{b}{0.22a}\right)^2\right]}$$

From that equation and an assumed value of 0.2 for $\nu$, Fig. 3 is constructed. We can compute the shear at face of column easily and multiply it by the chart value of $(dM_r/dr)/Q$ to obtain $dM_r/dr$ to be used in the bond equation

$$u = \frac{dM_r/dr}{\sum ojd}$$

Obviously the results are not precise. Assumptions of zero moment at $r = 0.22a$, round columns, and equal spans form the bases for the formula. In addition, the material is not homogeneous and the reinforcing bars are not radial. However, here is a starting point for selecting a better theory and for making tests to confirm or disprove it.*

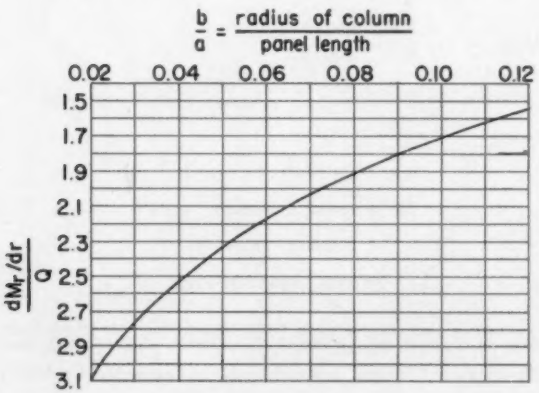


Fig. 3—Values of moment-shear ratio used to calculate bond stress in slabs

*Preliminary tests are in progress and indicate some confirmation of theoretical coefficients.

# Problems and Practices

*A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.*

*To some, the answers will seem simple and obvious; others may prove controversial.*

*All ACI members are invited to participate—either by submitting an inquiry, or even better, by telling JOURNAL readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.*

---

*Further discussion has been received on torsional stresses in spandrel beams of flat slabs designed according to ACI 318-56. The original question appeared in the Problems and Practices Section, ACI JOURNAL, Jan. 1959, p. 814.*

Concerning the torsional stresses occurring in the spandrel beams of flat slabs if these slabs are designed according to ACI 318-56, I would like to add the following discussion.

Although concrete members have the ability to relieve themselves of torsional stresses, and although both common practice and several building codes rely on this ability, I personally feel that in certain cases the combined shearing and torsional stresses should be investigated, and that a note to that effect should appear in the codes along with a table of allowable stresses.

In our office we have designed rigid building frames relying entirely on the torsional resistance of the spandrel beams, and we used as a guide for allowable torsional stresses the "Hellenic Technical Regulations (TEK 1950)." A free translation of the related paragraphs is:

"In the case where a member is loaded by torsional moment only, and the torsional shearing stress exceeds the allowable for unreinforced concrete, the total diagonal tension is to be taken by reinforcement.

"Torsional shearing stress in properly reinforced members shall not exceed the allowable shearing stress in properly reinforced concrete.

"The same regulations as above shall apply for the algebraic total of the shearing stresses that are caused by the simultaneous action of torsional and bending moment, with the only difference that allowable stresses are higher The member shall be checked also for each loading separately so that each separate stress shall not exceed the corresponding allowable stress for each loading."

## ALLOWABLE STRESSES

| Strength of concrete in 28 days ($f_c'$) | | | | |
|---|---|---|---|---|
| kilograms per sq cm | 120 | 160 | 225 | 300 |
| psi | 1710 | 2280 | 3200 | 4270 |
| **Shear without special reinforcement:** | | | | |
| (a) Maximum allowable stress without special reinforcement ($v_c$) | | | | |
| for slabs | | | | |
| kilograms per sq. cm, | 6 | 8 | 9 | 10 |
| psi | 85.4 | 113.8 | 128 | 142.2 |
| for other members | | | | |
| kilograms per sq cm | 4 | 6 | 7 | 8 |
| psi | 57 | 85.4 | 99.6 | 113.8 |
| (b) Maximum allowable stress in properly reinforced members. (Total stress to be taken by reinforcement.) ($v$) | | | | |
| kilograms per sq cm | 14 | 16 | 18 | 20 |
| psi | 199 | 228 | 256 | 284 |
| **Torsional shear of rectangular sections:** | | | | |
| (a) Maximum allowable stress without special reinforcement ($v_c$) | | | | |
| kilograms per sq cm | 4 | 5 | 6 | 7 |
| psi | 57 | 71.2 | 85.4 | 99.6 |
| (b) Maximum allowable stress in properly reinforced members. (Total stress to be taken by reinforcement.) ($v$) | | | | |
| kilograms per sq cm | 14 | 16 | 18 | 20 |
| psi | 199 | 228 | 256 | 284 |
| **Shear due to bending action simultaneously with torsion:** | | | | |
| (a) Maximum allowable stress without special reinforcement ($v_c$) | | | | |
| kilograms per sq cm | 6 | 8 | 9 | 10 |
| psi | 85.4 | 113.8 | 128 | 142.2 |
| (b) Maximum allowable stress in properly reinforced members. (Total stress to be taken by reinforcement.) ($v$) | | | | |
| kilograms per sq cm | 17 | 20 | 23 | 26 |
| psi | 242 | 284 | 327 | 370 |

The same regulations comment on maximum negative moments and minimum positive moments at the midspan of continuous slabs. The free translation of these comments is:

> *"Negative moments at midspan.* Because of the torsional resistance of the supporting concrete beams when placed integrally with continuous reinforced concrete slabs, it is allowed that these slabs be designed to resist half the theoretical value of negative moments at midspan when these moments are due to live load only."

This of course explains the common practice of providing negative reinforcing in slabs for a length equal to the quarter span even if the proportion of live to dead load indicates negative moment throughout the span.

The next paragraph is also based on the effect of torsion.

> *"Minimum positive moment at midspan.* If the theoretical maximum positive moment at the midspan of a continuous slab proves to be less than the positive moment resulting from fixed-end conditions, the slab at midspan is to be proportioned for this last condition."

Concerning the restraint induced by a spandrel beam the same regulations comment:

> *"Restraint of end support.* For the establishment of the maximum positive moment for the end span of a slab, such an amount of restraint of the exterior support may be taken into account as can be proved certain and safe by analysis."

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Senior Design Engineer
Shore and Moffat
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* * * * *

*Q. Further comment is offered on the question: What is the detrimental influence of mica in concrete aggregates and the critical percentage if limits have been established?**

*A.* Large amounts of mica in a rock tend to indicate igneous or metamorphic origin involving original crystallization at great pressure and high temperature. The mica may be carried over into sedimentary formations, but due to the weak structure of the mica, there would not ordinarily have been much modification of the original mineral composition of the rock if much mica remains in the sediment. This latter would also be generally true of a fine aggregate containing large amounts of mica. Hence the mica, in addition to being structurally weak itself, tends to indicate the presence of other minerals formed under the same circumstances.

Such minerals, having achieved forms of stability under conditions at depth in the earth's crust, may then be expected to display relative instability at the earth's surface. In the presence of the active chemical agents of portland cement, a relatively high tendency to alter to other forms may be expected. This suggests the possibility of undesirably high aggregate reactivity of unknown character.

Practically speaking, these considerations reinforce the final paragraph of the original answer.*

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Head of Civil Engineering
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Northfield, Vt.

*Concrete Briefs section, Problems and Practices, ACI JOURNAL, V. 30, No. 5, Nov. 1958 (*Proceedings* V. 55), p. 658.

**The Problems and Practices section is an open forum on everyday topics. JOURNAL readers are invited to contribute their solutions to problems that occur in the field or office—whether common or unusual.**

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**Items appearing in this section can vary from 100 to 1000 words. It is an ideal place for those practical items generally considered too short to be submitted for publication, but containing material of potential importance to a great many.**

# CURRENT REVIEWS

## of Significant Contributions in Foreign and Domestic Publications

## Construction

### Waterfront structure design for varying conditions

WILLIAM C. STEVENS and JOHN S. WILSON, *Proceedings*, ASCE, V. 84, WW 3, May 1958, pp. 1639-1–1639-31

AUTHORS' SUMMARY

Review of the different designs of pier and wharf structures constructed by the Port of New York Authority during the past 10 years. Effects of site conditions and functional requirements on each of six major projects are discussed and the resulting construction described.

### Field inspection of building construction

THOMAS H. MCKAIG, F. W. Dodge Corp., New York, 1958, 352 pp. $9.35

Scope of book is broad, covering all phases of construction and many different methods and trades. Following a discussion on the job of the inspector, the construction project is followed through the preliminary, foundation, structural framing, intermediate, and finishing stages. A separate chapter is devoted to inspecting concrete work; the predominant source reference being the *ACI Manual of Concrete Inspection*.

### Structural features of the new Binns store in Middlesborough

F. R. BULLEN, *The Structural Engineer* (London), V. 35, No. 11, Nov. 1957, pp. 408-419

Reviewed by C. P. SIESS

Describes design and construction features of large department store including: reinforced concrete foundation piles; reinforced concrete retaining walls around deep basement; continuous rows of bored piles for underpinning adjacent structures; erection of steel frame; and elliptical reinforced concrete stair extending five stories of 15 ft each with intermediate supports between stories. Provision was made for post-tensioning stair to reduce deflection but was not needed and not used.

### Scaffolding—your guide to major types, specifications, applications

*Building Construction Illustrated*, V. 26, No. 1, Jan. 1959, pp. 54-58

General specifications for 14 types of frame scaffolding arranged in tabular form. Includes load capacities, maximum height, standard panel widths, type, size, and material of leg. Also includes a list of safety rules and regulations.

## Construction Techniques

### Composite construction

FELIX J. SAMUELY, *The Reinforced Concrete Review* (London), V. 3, No. 8, 1955, pp. 477-499; discussion pp. 499-510

Reviewed by WILLIAM R. LORMAN

Concerned with the practical aspects of composite construction which combines the advantages of prefabricated materials, such as steel and precast concrete, and the advantages of in-situ construction. These advantages are presented and examined thoroughly as six items in the first half of the paper. The second half is concerned with examples of three main types of composite construction involving the following combinations: (1) precast and in-situ concrete, (2) prestressed and in-situ concrete, and (3) structural steel and in-situ concrete.

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For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

### Safe anchorages in concrete

F. E. Jones, *Rubber Developments* (London), V. 11, No. 4, 1958, pp. 131-132

Attaching materials such as wood floors, vertical cladding systems, or stone facing to a concrete structure is done by means of slots or square-cut metal channels which are formed in the concrete during its placement. Using rubber molds to form these channels, it is possible, after the concrete is hardened, to withdraw the flexible rubber formers and reuse them. Metal anchors which are inserted into the shaped concrete slots are secured with grout which forms a better bond with the concrete slot than it does with its metal counterpart.

## Dams

### Design and construction of certain temporary works, Upper Silent Valley Reservoir

W. J. Shirley, *The Structural Engineer* (London), V. 35, No. 10, Oct. 1957, pp. 368-381

Reviewed by C. P. Sikes

Describes construction features for large dam including: a large steel derrick gantry of 400-ft span; a timber and steel set of aggregate bins of 900-ton capacity; and a precast concrete screening and aggregate storage bin of 1800-ton capacity.

## Design

### Reinforced concrete in coast protection works

J. A. Lewis, *The Reinforced Concrete Review* (London), V. 3, No. 10, 1955, pp. 697-707; discussion pp. 707-715

Reviewed by William R. Lorman

Presents the economic and technical advantages of using reinforced concrete for sea walls; also considers the "retaining wall" aspects of sea walls of reinforced concrete. Among the technical features included are design factors which are divided into three groups: (1) wave pressure; (2) beach level, configuration, and material; and (3) ground formation and material behind the wall. Also presents the practical aspects of designing reinforced concrete walls subjected to varying conditions. This is followed by a brief section pertaining to the disadvantages of employing reinforced concrete in jetties; prestressed concrete is unwarranted in view of potential deterioration by abrasive forces.

Discussion covers suction damage, concrete durability, mix proportioning, and construction joints.

### Sandwich-type structures made up of connector-joined panels (in Swedish)

Åke Holmberg, *Nordisk Betong* (Stockholm), V. 2, No. 4, 1958, pp. 377-388

Reviewed by Margaret Corbin

Deals with structural units consisting of two concrete panels with a mineral wool mat between them. A method is proposed for designing the connectors which hold the panels together. This is followed by calculations of deflection, mean shearing stress, moments and forces due to a vertical load acting on a supported panel; to a vertical load acting on a bearing panel; to a uniformly distributed load acting at right angles to the unit; and to a difference in temperature. An example is worked out.

### Load-factor methods of designing reinforced concrete

F. G. Thomas, *The Reinforced Concrete Review* (London), V. 3, No. 8, 1955, pp. 511-555; discussion pp. 555-569

Reviewed by William R. Lorman

This paper is concerned principally with ultimate strength features of reinforced concrete. The introductory portion includes definitions of the three primary principles.

A review of current knowledge covers: safety risks, loads in structures of various types, basis for strength calculations (concrete deformations, steel deformations, and deformational variations across any cross section), axially and eccentrically loaded columns, helically reinforced columns, beams, two-way slabs, interaction between slabs and beams, and continuous beams and frameworks.

The concluding portion is concerned with suggestions for modifying strength formulas used in designs that offer sufficient load factors against not only structural collapse but also against abnormal cracking and deformations in the structures.

The discussion ranges in subject from design calculation coefficients to statistical techniques for calculating failure probability.

### Practical aspects of ultimate strength design

ALFRED L. PARME, *Proceedings*, ASCE, V. 84, ST 5, Sept. 1958, pp. 1757-1–1757-22

AUTHOR'S SUMMARY

Time-saving design charts for the rapid selection of the critical-load-factor combination are presented. Also includes studies comparing the loads and area of reinforcement as obtained by the ultimate strength procedure to that obtained by conventional methods.

### Thermal considerations in the design of concrete shields

HAROLD S. DAVIS, *Proceedings*, ASCE, V. 84, ST5, Sept. 1958, pp. 1755-1–1755-25

AUTHOR'S SUMMARY

Behavior and properties of concrete structures for shielding atomic power plants are discussed briefly. Methods are presented for estimating thermal effects associated with linear and nonlinear distributions of temperature, nuclear heating, and shield cooling. These data and procedures are subsequently used for analyzing several design problems and thermal criteria.

### Slopes and deflections of beams

R. J. CORNISH, *The Reinforced Concrete Review* (London), V. 3, No. 8, 1955, pp. 587-594

Reviewed by WILLIAM R. LORMAN

The procedures set forth in this paper are claimed to be simpler and faster than Newmark's methods (for determining slopes and deflections of structural members) which appear in V. 3, No. 5 (1954) of this periodical. The analytical procedures are presented in four problems two of which are from Newmark's paper. An appendix contains proof of the Simpson's Rule method. Four tables and two figures are used to demonstrate the procedural steps.

### Reinforced concrete fundamentals

PHIL M. FERGUSON, John Wiley and Sons, Inc., New York, 1958, 604 pp., $9.50

Designed for use as a text in an introductory course in reinforced concrete, this book covers topics not previously a part of an undergraduate program. Emphasis is placed on ultimate strength theory and behavior of members under load. These emphases are made because of the author's feeling that while specifications change, failure patterns represent permanent knowledge. However, text is written so that teachers who wish may defer the study of ultimate strength theory.

Topics covered include beam analysis by ultimate strength theory and conventional theory, axially and eccentrically loaded columns, and cantilever retaining walls. Four chapters are devoted to slabs—one-way, two-way, and flat slabs are discussed, and one of the chapters treats yield-line theory for slabs. There are also chapters on footings and prestressed concrete analysis. Deflections, and frame analysis by moment distribution and by limit design are discussed in appendices.

## Materials

### Influence of sand containing humic compounds on strength of concrete (in Swedish)

TOIVO KARTTUNEN and TENHO SNECK, *Nordish Betong* (Stockholm), V. 2, No. 4, 1958, pp. 389-396

Reviewed by MARGARET CORBIN

The sodium hydroxide test for organic impurities ignores the chemical nature of such impurities. Therefore, a sand which yields a negative reaction can be potentially undesirable. The paper reviews some investigations dealing with the mechanism of the action of humus. Forsen has shown that humic substances may act as accelerators in the presence of alkalies. The author believes that this effect may be due to the effect of inorganic salts present on the surface of humic sand. It is pointed out that a more reliable test method is needed.

### Pozzolanic properties of Washington State pumice and pumicites

E. N. KLEMGARD, *Bulletin* 242, Washington State Institute of Technology, State College of Washington, 1958

HIGHWAY RESEARCH ABSTRACTS Sept. 1958

Nearly all of the Washington pumicites and pumices have positive pozzolanic properties. Their pozzolanic activity can be appreciably enhanced by calcination. The properties of pozzolanic materials for 21 deposits within the state have been determined with respect to grindability, density, surface area, available alkalies, drying shrinkage, and compressive strength. This information

should be useful for those concerned with the economics and mix proportioning of concrete for massive structures at various locations in the state.

### Use of sawdust as concrete aggregate (in Rumanian)

M. A. SERBAN, *Constructiilor si a Materialelor de Constructii* (Bucharest), V. 10, No. 7, July 1958, pp. 373-379

Reviewed by J. J. POLIVKA

Thorough discussion of possibilities of using sawdust with sand, slag, ashes, and gravel for various purposes, as substitute for other building naterials.

### Wet grinding of cement raw materials

J. RICHARD TONRY, *Pit and Quarry*, V. 51, No. 8, Feb. 1959, pp. 93-97

AUTHOR'S SUMMARY

Wet raw grinding with the operation of circuits using classifiers is discussed. Various classifiers are being used in the cement industry to increase the production rate of grinding equipment and to eliminate oversize particles from slurry.

## Pavements

### New membrane claimed to revolutionize concrete laying practice

*Highways and Bridges and Engineering Works* (England), V. 26, No. 1261, Oct. 1, 1958, p. 8

HIGHWAY RESEARCH ABSTRACTS Feb. 1959

"Lubrithene" is a thermoplastic membrane material (with a high-slip finish, $f = 0.25$) that is laid on the base prior to the placement of concrete. This slippery finish permits the pavement slab to move or "float" horizontally, which limits early uncontrolled cracking thus allowing a reduction in 28-day compressive strength from say 6500 psi to 2500 psi.

### Concrete road construction in nine European countries (in Dutch)

VAN DER BURGH, DUTRON, and STREIT, *Cement* (Amsterdam), V. 10, No. 17-18, June 1958, pp. 725-734

Reviewed by JOHN W. T. VAN ERP

Article is one of the papers presented at the International Concrete Road Congress, Rome, 1957. Gives numerous examples and information on: concrete mixes; curing; slab thickness and loads; monolithic or layer construction; reinforcement, conventional or prestressed; amount of reinforcement per unit area; testing by drilled cores, conventional or nondestructive; joints, and joint spacing. Conclusion is for the increase of prestressing, which has proved quite economical and satisfactory for road and airport pavements.

## Precast Concrete

### Calcium chloride as accelerator for precast concrete (El cloruro cálcico como acelerador en la prefabricación de hormigón)

J. CALLEJA CARRETE, *Bulletin* No. 185, Instituto Técnico de la Construcción y del Cemento, Madrid, 1958, 38 pp.

Reviewed by ROGER DIAZ DE COSSIO

After a brief historical introduction, the paper describes with great detail an experimental investigation made to determine, for given local conditions and materials, the optimum amount of calcium chloride necessary to obtain a strong mortar in a short time. The optimum dose was found to be 3 percent, and it was successfully applied to the fabrication of precast concrete window frames.

### Manufacture of concrete masonry units

WILLIAM GRANT, 2nd Edition, Concrete Publishing Corp., Chicago, 1959, 261 pp., $5

The original edition has been revised and enlarged considerably. A practical handbook on the manufacture of concrete masonry units, the recommended practices contained in it should serve as a guide toward accomplishing the improved control desirable in present day concrete masonry manufacture.

Sections are devoted to cement and aggregates, including properties, testing, grading, and proportioning; admixtures; mixing and processing concrete; kilns; curing; cracks in block walls; prefabricated floor systems; steam boilers; fuels and combustion; selection of block machine; and specifications and testing.

### Tests of precast expanded clay concrete slabs reinforced with prestressed elements of dense concrete (in Russian)

J. G. SUNGATULLIN, *Beton i Zhelezobeton* (Moscow), July 1958, pp. 270-273

Reviewed by D. WATSTEIN

Two series of tests were made on expanded clay concrete slabs 6.3 in. deep with a span of 11.8 ft. One series of slabs was reinforced

with 2 in. square prestressed prisms of dense concrete and the other series with 1.2 x 8-in. prestressed planks. The prestressed prisms were spaced about 14 in. apart, while the prestressed planks formed a continuous lower surface of the slabs. The paper presents the results of sustained load tests and short term tests. The properties investigated include flexural rigidity, strain distribution along the entire depth of the slabs, formation of cracks, and resistance to shear.

Composite slabs of the type investigated compare favorably with monolithically cast dense concrete prestressed elements with respect to expenditure of steel and high strength concrete, and dead weight. The fabrication process is reportedly quite simple and requires no elaborate installation.

## Prestressed Concrete

### Prestressed concrete beams: tests on beams with 0.276-in. wire

*The Reinforced Concrete Review* (London), V. 3, No. 8, 1955, pp. 595-600

Reviewed by WILLIAM R. LORMAN

Brief description of static and fatigue tests conducted by the Civil Engineer's Department of the British Railways (Eastern Region). The test results corroborated the feasibility of 0.276-in. diameter smooth steel wire for pretensioning of high-strength concrete beams having various lengths up to 22 ft.

### The Gifford-Udall-CCL system and some recent developments in prestressed concrete

E. W. H. GIFFORD, *The Reinforced Concrete Review* (London), V. 3, No. 10, 1955, pp. 735-771; discussion pp. 771-779

Reviewed by WILLIAM R. LORMAN

This prestressing system was developed to decrease the labor and equipment costs and to facilitate the general adoption of prestressing as a construction technique especially with regard to the smaller type structures. One of the prerequisites was that "... one man should be able to carry out the complete stressing operation with a single tool that he could carry by himself." By limiting the combined jack and pump weight to a maximum of 56 lb, the maximum force attained may be slightly beyond 13,000 lb; the maximum size wire may be 0.276 in. (7 mm) diameter; the CCL anchor grip is considered the most compact for single-wire work. The system was employed initially in 1953.

The paper furnishes details concerning: the grip, jack, cable and anchorage, curved stressing tube (for situations where space behind the anchorages is insufficient to operate the jack), edge- and groove-type anchorages, circular tank anchorage, stressing stool (for insertion of shims between anchor grip and bearing plate), and double-ended spring-loaded grip. One section consists of two groups of examples; one shows construction jobs that emphasize economy and the second pertains to jobs that incorporate unique prestressing features. Footbridges, frames, trusses, beam systems, an industrial building, and a research laboratory are covered in the first group; cantilever roof beams, church, residential structure, and underpinning work are featured in the second group. Appendix I pertains to tests on the CCL grips and Appendix II contains test and computational data with regard to a 46-ft span highway bridge beam.

The discussion centers about operational details that relate to: the stressing stool, bond of the wire, hydraulic jack pressure indicators, the use of plasticizers in the grout, types of anchorage, and efficiency of operators.

The reader will find that this paper is interesting and well-written.

### Progress in precast prestressed concrete (in Dutch)

STUVO COMMITTEE, *Cement* (Amsterdam), V. 10, No. 15-16, Apr. 1958, pp. 641-650

Reviewed by JOHN W. T. VAN ERP

In this particular field the joints made on the job are the most critical details of construction; their development from the simplest grouted beam-column joint acting as a hinge, into the more elaborate rigid joints are given. Different stages in the development of the rigid joints are: continuity by mild steel grouted in, continuity by mild steel in cast-on-the-job intermediate elements forming a combined prestressed and conventional structure, and continuity by field stressed units. Many examples are given of bridges with joints at points of inflection. These joints are often formed by prestressing laterally into gap-bridging elements.

### Creep and shrinkage measurements of prestressed concrete structures (Ergebnisse von Kriech- und Schwindmessungen an Spannbetonwerken)

ULRICH FINSTERWALDER, *Beton- und Stahlbetonbau* (Berlin), V. 53, No. 5, May 1958, pp. 136-144
Reviewed by VALDIS LAPSINS

Gives new results of structures tested, supplementing information previously published and contained in *Beton- und Stahlbetonbau* of January, 1955. Results of measurements verify that actual behavior of structures agrees with theoretical computations.

### Safe stresses in prestressed concrete (Résistance et sécurité du béton précontraint)

F. DUMAS, *Travaux* (Paris), V. 42, No. 289, Nov. 1958, pp. 977-985

A general review containing recommendations for design, materials, and construction requirements to achieve maximum safety in prestressed concrete structures.

### Recent prestressed structures in the Netherlands (in Dutch)

A. S. G. BRUGGELING, *Cement* (Amsterdam), V. 10, No. 15-16, Apr. 1958, pp. 635-638
Reviewed by JOHN W. T. VAN ERP

Examples of bridges, factory buildings, and roads are given. Bridges are mostly continuous. Also example of an arch bridge, with a high arch, without lateral bracing of the arches, and with a prestressed road slab.

Good results are also reported of prestressed road slabs where joints are reduced to a bare minimum.

### Experimental study of prestressed concrete power transmission poles (in Russian)

A. D. LIEBERMAN and O. M. RUBACH, *Beton i Zhelezobeton* (Moscow), No. 11, Nov. 1958, p. 421-424
Reviewed by D. WATSTEIN

Three types of prestressed transmission poles designed to carry 6 to 10 kilovolts were recently investigated. These poles were 40 ft long and tapered from 9.5 x 13.4 in. at base to 5.5 x 7.9 in. at the top. One type of poles had an I-section with suitable diaphragms, while the other two types had solid sections and were perforated with oval or rectangular openings. The perforated poles of solid section were preferable from standpoint of concreting in that the forms could be filled in a single operation. The poles containing rectangular openings did not develop sufficient resistance to shear. The poles containing oval openings were recommended for adoption by the industry.

### Calculation of prestressed construction in building erection methods with particular attention to shrinkage and creep (Die Berechnung von Spannbetonkonstruktionen in Montagebauweise bei besonderer Berücksichtigung des Betonschwindens und -kriechens)

HERRMANN RUHLE, *Proceedings*, Second Congress of Fédération Internationale de la Précontrainte, Amsterdam, 1955 (published 1958), pp. 353-360

Results of theoretical studies based on extension of the elastic theory to plastic deformations emphasize that if concrete is added after assembly of precast prestressed units to give the final form of the structures, stresses will occur due to the different phenomena of shrinkage and creep in the two concretes which will affect the prestress.

## Properties of Concrete

### Hysteresis in length changes of concrete undergoing wetting and drying cycles (in Russian)

S. V. ALEXANDROVSKY, *Beton i Zhelezobeton* (Moscow), No. 9, Sept. 1958, pp. 344-346
Reviewed by D. WATSTEIN

Shrinkage and expansion studies were made on 5 x 5 x 11-cm prisms which were coated with paraffin on the sides and were permitted to dry only at the ends.

Author performed extensive measurements of shrinkage and expansion and the corresponding changes in the moisture content of the specimens. The length changes plotted against the changes in moisture content show characteristic hysteresis loops which become stabilized with time and approach a constant pattern. At the same time, the lower and upper limits of the length of the specimens are reduced with time.

Author cites the theory of Sheikin on the gel structure of the cement paste as being in accord with the observed moisture-deformation relationship.

## Improving the water repellency of hardened concrete

*Bulletin* No. 197, Highway Research Board, Nov. 1958, $0.50

### Scaling resistance of concrete improved through silicones

F. J. Mardulier, pp. 1-12

Discusses factors affecting scaling, field and laboratory tests, mechanics of silicone protection, hypothesis of silicone action, and depth of penetration, and presents some limited conclusions.

### New York State's experience in use of silicones

H. B. Britton, pp. 13-23

Discusses investigations of silicone application on field installations. Laboratory studies were also made.

## How to utilize dynamic elastic modulus of concrete (in Japanese)

Yoshiro Higuchi, *Journal*, Japan Society of Civil Engineers (Tokyo), V. 43, No. 7, July 1958, pp. 19-24
Reviewed by Kiyoshi Okada

Surveying the available information on dynamic modulus of elasticity, Poisson's ratio, and logarithmic decrement obtained from concrete specimens, author concludes that the dynamic modulus of elasticity determined by resonnant-sonic-flexural frequency is most suitably used for testing durability of concrete, while that by resonnant frequency of longitudinal wave is most effectively used for the quality control of concrete.

## Effect of early freezing upon basic properties of quick setting concrete (in Russian)

S. N. Korotkov, *Bulletin Stroitelnoi Tekhniki* (Moscow), V. 14, No. 8, Aug. 1957, pp. 15-18
Reviewed by Herbert E. Kebe

Presents research data on investigations of the effect of early freezing on reinforced concrete samples made by using quick setting and high strength cements. Studies consider the effect of early freezing on (1) final strength, (2) bond between the concrete and the steel reinforcement, and (3) watertightness. Concrete resistance to frost and time factor are also being studied. The results suggest that if quick setting concrete is kept for the first 3 days at normal temperatures, frost effects are negligible on its basic properties.

## Effect of initial curing temperature on the strength of mortar and concrete (in Japanese)

Shunsuke Takano and Akio Yanagawa, *Semento Gijutsu Nenpo*, V. 7, 1953, pp. 225-229; Part II, V. 8, 1954, pp. 298-304
Ceramic Abstracts
Oct. 1958 (Y. Suzukawa)

Mortar specimens made with normal portland cement, having temperatures of 5, 20, 35, and 50 C at the end of mixing were cured at these temperatures for 1.5, 3, 6, and 24 hr, respectively, and then cured moist or wet at 20 C until the time of test. Mortar strength at 3 days increases with rising initial curing temperature, but the lower the initial curing temperature, the greater is the strength development at later ages. An initial curing period of 1.5 to 24 hr has little influence on the strength of mortar. The strength of neat cement paste and concrete cured under the same conditions is similar to that of mortar. These results agree with those of thermogravimetric analysis of hydrated cement pastes cured initially at 5 to 50 C for 24 hr.

Part II: The same tests were made on a normal, a rapid hardening, and a moderate heat cement. The effect of initial curing temperature on strength development at later ages of the rapid hardening and moderate heat cements is not so marked as in the case of the normal cement. The relation between the amounts of combined water and the compressive strength of neat paste was determined, and differential thermal analysis was made. No explanation is given, however, for the difference in the effect of initial curing temperature on strength development at later ages according to the kind of cement.

## Mechanisms by which air-entraining agents affect viscosities and bleeding properties of cement pastes

G. M. Bruere, *Australian Journal of Applied Science* (Melbourne), V. 9, No. 4, Dec. 1958, pp. 349-359
Author's Summary

Measurements were made of the effects of a series of surface-active agents on viscosities, bleeding rates, and bleeding capacities of cement pastes both in the absence of entrained air and in the presence of varying amounts of entrained air.

The results show that the factors affecting the viscosities of cement pastes containing air-entraining agents are the effect of the

agent on the adhesion of air bubbles to cement grains, the effect of the agent on the interparticle attraction between cement grains, and the presence of air bubbles themselves.

The factors affecting bleeding rates and capacities of cement pastes containing air-entraining agents are the effect of the agent on degree of flocculation of cement grains and the presence of air bubbles.

It is concluded that both the adsorption characteristics of the agent on cement and the air bubbles produced by the agent have important influences on the properties of fresh air-entrained concrete.

## General

### Testing of prestressing materials and concrete control on the Northern Illinois Toll Highway

JOSEPH J. WADDELL, *ASTM Bulletin*, No. 234, Dec. 1958, pp. 35-40

Describes the testing and inspecting methods used in construction of the Northern Illinois Toll Highway. The use of prestressed concrete girders and hollow concrete piles for over 200 bridges necessitated new testing procedures.

Discusses the techniques developed or modified for testing fabric pads for bridge bearings, making tension tests of 7-wire prestressing strand, control of stressing operations when both straight and deflected strands were used, and making compressive strength tests of zero slump concrete.

### Theory of concrete (in Rumanian)

V. NICOLAU, S. ABRANOVICI, S. ENESCU, N. MIHAIL, G. STANCULESCU, and R. ZVIBEL, V. 1, State Edition of Architecture and Construction, 1954, 316 pp., 18 Lei; V. 2, Technical Edition, Bucharest, 1957, 516 pp., 24.5 Lei

Reviewed by J. J. POLIVKA

One of the most thorough publications on concrete as a building material. Discusses its ingredients and their properties, cementing materials and admixtures, recent developments, improvements, and applications, especially with steel reinforcement. There are 18 pages devoted to the history of cementing materials used by the Romans in the construction of aqueducts, bridges, and other massive work. Both volumes present in precise form practically all important knowledge, research, publications, etc., available with 128 references. Theoretical methods of design are thoroughly compared with experimental stress analysis, including photoelastic tests.

V. 2 contains chapters on special structural elements and members, advantages and disadvantages of their design, assembly and execution, factors of safety, and possibilities of failures. Great part of the literature to which references are made, is of Rumanian, Russian, and Yugoslav origin, but does not overlook important work and publications in the other European countries and in the Americas.

### Quality control for large highway projects

EDWARD A. ABDUN-NUR, *Proceedings*, ASCE, V. 84, HW 2, May 1958, 1626-1–1626-10

Large highway projects need quality control of not only building materials but also of construction methods and operations.

A proposed criteria for such a control requires standards of quality, specifications that permit attainment of such standards, and training and organizing a supervisory inspection force.

With an intelligent application of a workable control, the provisions of the plan can be converted into reality. The author contends that this can lower costs, improve quality, speed operations, and provide a highway which will give the utmost uninterrupted use.

### Crushing and grinding

Department of Scientific and Industrial Research (England), Her Majesty's Stationery Office, London, 1958, 425 pp., 35s.

A carefully selected list of 2800 literature references, abstracted or annotated, prepared under the auspices of the Department of Scientific and Industrial Research and the Institution of Chemical Engineers. An interesting and valuable feature is the collection of short reviews on the current state of knowledge on the fundamental aspects of crushing and grinding, problems of breakage and structure of coal, methods of particle size analysis, grinding problems in various specialized industries, and fire and explosion hazards in crushing and grinding operations. Author and subject indices add to the usefulness of the compilation.

# ACI News Letter

ol. 30, No. 11 JOURNAL of the AMERICAN CONCRETE INSTITUTE May 1959

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News Letter Contents

May 1959

On the Cover—A 160 ft high concrete bell tower, one of the highest free standing concrete towers in the world, is part of an extension to Westminster Abbey, Ltd., a Benedictine monastery at Mission City, B. C., Canada. Acoustical problems received primary consideration in the tower's design. The vibration of the bells required careful study and analysis to limit the allowable movement at the bell level to 1/50 (0.02) in. Since the concrete had to be hauled 30 miles in transit-mix trucks, a retarder was used to prevent the premature stiffening in the trucks and cold joints during placement. Decorative inserts were cast at the site. Joints in the tower, above the section containing the bells, are lined with a flexible permanent mastic.

Gardiner, Thornton, Gathe and Associates were the architects and Read, Jones, Christoffersen were the structural engineers. Doyle Construction Co., Ltd., served as general contractor; ready-mixed concrete was furnished by Deeks McBride, Ltd. All are situated in Vancouver, B. C. —*Photo courtesy Sika Chemical Corp.*

Four Committees Discharged..... 3

Technical Committee Appointments..... 5

Hunter College Library Roof..... 7

Positions and Projects.....11

Building Research in Australia.....14

Bridge Deck Forming.....17

Looking Ahead.....29

Honor Roll.....31

New Members.....32

Tools, Materials, Services..34

List of Advertisers.....40

# Board of Direction Discharges Four ACI Technical Committees

Four ACI Technical Committees, having accomplished their respective missions, were discharged by the Institute Board of Direction at its February 22 meeting in Los Angeles: Committee 210, Resistance to Erosion in Hydraulic Structures; Committee 317, Reinforced Concrete Design Handbook; Committee 617, Specifications and Recommended Practice for Concrete Pavements and Bases; and Committee 711, Precast Floor Systems for Houses.

## Committee 210, Resistance to Erosion in Hydraulic Structures

Originally organized in 1940, this committee was reorganized in 1947 and given the assignment of studying all types of erosion occurring in hydraulic structures.

The committee report, "Erosion Resistance of Concrete in Hydraulic Structures," was published in the November, 1955, JOURNAL. This report was concerned mainly with the physical erosion of concrete in hydraulic structures resulting from particles carried by flowing water, and from pitting resulting from cavities forming and collapsing in water flowing at high velocities. Disintegration of concrete by chemical attack was also discussed. Materials, mix proportions, and construction procedures which will make concrete more resistant to such erosion were covered.

D. S. Walter, regional engineer, U. S. Bureau of Reclamation, Boise, Idaho, was chairman of the committee at the time of discharge.

## Committee 317, Reinforced Concrete Design Handbook

Organized in 1939, Committee 317, was reactivated in 1953 for the purpose of bringing up to date the ACI special publication *Reinforced Concrete Design Handbook* published in 1939.

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**ACI Publications Department**

The Committee revised the handbook to conform with the latest codes and practice. The outline of the handbook was maintained with the revisions directed at: extension of existing tables to include higher strength of concrete and steel; deletion of sections dealing with obsolete codes; minor improvements in figures and charts; and inclusion of new material on columns and footings.

The second edition was made available late in 1955 and to date over 22,000 copies have been ordered from ACI headquarters.

Thor Germundsson, staff advisor, Portland Cement Association, Chicago, served as committee chairman.

## Committee 617, Specifications and Recommended Practice for Concrete Pavements and Bases

This committee was organized in 1937 to prepare and maintain a current specification for materials and construction of concrete pavements and also concrete bases used to support surface courses of other materials. The specification complements the recommendations for design by Committee 325, Structural Design of Concrete Pavements for Highways and Airports.

The committee was responsible for compiling the standard, "Specifications of Concrete Pavements and Concrete Bases (ACI 617-58)," superseding ACI 617-51. The standard was published in July, 1958, JOURNAL, setting forth specifications which apply to construction of portland cement concrete pavements and bases under normal conditions for both highways and airports.

Sources and acceptance standards for materials were given, as well as procedures for materials testing and concrete strength testing to be used as a basis for proportioning mixtures. Specifications for the materials and construction of soil foundations for concrete pavements and concrete bases were included. Provision was made for use of foundations stabilized by a cementing agent, but materials and construction of such were considered beyond the scope of the specification.

Materials, dimensions, setting, and removal of forms were treated. Construction methods were specified for forming joints, installation of joint seal and load transfer devices, and placing of reinforcement. Concrete proportioning based on design for minimum strength was covered in detail.

Other sections covered production of high-early-strength concrete and the handling and mixing of materials. Detailed requirements for placing, finishing, and curing of pavement concrete were given.

H. F. Clemmer, engineer of materials, Department of Highways, Government of the District of Columbia, Washington, D. C., was chairman of the committee at the time of discharge.

## Committee 711, Precast Floor Systems for Houses

Committee 711 was organized in 1934 and assigned the task of preparing current recommendations for the use of factory produced precast concrete floor and roof units.

The latest report of this committee, "Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)" superseding ACI 711-53, was published in the July, 1958, JOURNAL. Minimum standard requirements were presented for single units or multiple element assemblies, to be used in conjunction with ACI 318-56, covering materials, design principles, manufacture including curing and handling, testing of completed units, installation plans, and special provisions for holes and openings in members. Such problems as dimensions, allowable deflection, structural concrete topping, reinforcement anchorage and location, and use of lightweight concrete were covered in a design chapter.

Leo M. Legatski, professor, Department of Civil Engineering, University of Michigan, Ann Arbor, served as chairman of the committee.

# Technical Committee Appointments

Five new committee chairmen were appointed for 3-year terms, effective Feb. 22, 1959, with the former committee chairmen remaining as members of the respective committees.

**Committee 207, Properties of Mass Concrete**

S. D. Burks, Dewey and Almy Chemical Co., San Leandro, Calif. replaces I. L. Tyler, Portland Cement Association, Chicago.

**Committee 208, Bond Stress**

David Watstein, National Bureau of Standards, Washington, D. C., replaces C. P. Siess, University of Illinois, Urbana.

**Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete**

Adrian Pauw, University of Missouri, Columbia, replaces J. Neils Thompson, University of Texas, Austin.

**Committee 611, Inspection of Concrete**

M. R. Smith, Office, Chief of Engineers, Department of the Army, Washington, D. C., replaces Lewis H. Tuthill, California State Department of Water Resources, Sacramento.

**Committee 613, Recommended Practice for Proportioning Concrete Mixes**

Clayton L. Davis, Universal Atlas Cement Division, U. S. Steel Corp., New York, replaces William A. Cordon, Utah State University, Logan.

The following reappointments as technical committee chairmen were approved because of committee work in progress:

**Committee 323 (ACI-ASCE), Prestressed Reinforced Concrete**

Thor Germundsson
Portland Cement Association
Chicago, Ill.

**Committee 324, Precast Reinforced Concrete, Thin Sections**

A. Amirikian
Bureau of Yards and Docks
Department of the Navy
Washington, D. C.

**Committee 325, Structural Design of Concrete Pavements for Highways and Airports**

E. A. Finney
Michigan State Highway Department
East Lansing, Mich.

**Committee 326 (ACI-ASCE), Shear and Diagonal Tension**

Charles S. Whitney
Ammann and Whitney
New York, N. Y.

**Committee 331, Structures of Concrete Masonry Units**

S. H. Westby
Portland Cement Association
Chicago, Ill.

**Committee 605, Hot Weather Concreting**

Stanton Walker
National Sand and Gravel Association and National Ready Mixed Concrete Association
Washington, D. C.

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**ACI Publication Department**

**Committee 609, Compaction of Concrete by Mechanical Means**

J. J. Waddell
Knoerle, Graef, Bender and Associates, Inc.
Chicago, Ill.

**Committee 614, Recommended Practice in Measuring, Mixing, and Placing Concrete**

Lewis H. Tuthill
California State Department of Water Resources
Sacramento, Calif.

**Committee 621, Aggregates**

William R. Waugh
Office, Chief of Engineers
Department of the Army
Washington, D. C.

**Committee 622, Formwork for Concrete**

Harry Ellsberg
Giffels and Rossetti
Detroit, Mich.

Listed below are committee members who have recently accepted appointments to ACI technical committees. Included are *new* appointments only.

**Committee 116, Nomenclature**

Clayton L. Davis
Universal Atlas Cement Division
U. S. Steel Corp.
New York, N. Y.

Raymond E. Davis, Jr.
Intrusion Prepakt, Inc.
Cleveland, Ohio

**Committee 208, Bond Stress**

A. C. Bianchini
University of Illinois
Urbana, Ill.

James Chinn
University of Colorado
Boulder, Colo.

**Committee 215, Fatigue of Concrete**

Lawrence D. Childs
Portland Cement Association
Chicago, Ill.

**Committee 318, Standard Building Code**

M. P. Van Buren
Di Stasio and Van Buren
New York, N. Y.

**Committee 336, Combined Footings**

Norman Brunkow
Graham, Anderson, Probst and White
Chicago, Ill.

Fritz Kramrisch
Albert Kahn Associated Architects and Engineers, Inc.
Detroit, Mich.

**Committee 338, Torsion**

Paul Zia
University of Florida
Gainesville, Fla.

**Committee 340, Ultimate Strength Design Handbook**

George Winter
Cornell University
Ithaca, N. Y.

**Committee 611, Inspection of Concrete**

Joseph Di Stasio, Jr.
Di Stasio and Van Buren
New York, N. Y.

**Committee 613, Recommended Practice for Proportioning Concrete Mixes**

Paul Klieger
Portland Cement Association
Chicago, Ill.

**Committee 624, Portland Cement Plaster**

Raymond L. Blaine
National Bureau of Standards
Washington, D. C.

Harold Gatchell
Office, Chief of Engineers
Department of the Army
Washington, D. C.

Rolf T. Retz
California State Division of Architecture
Los Angeles, Calif.

**Committee 714, Concrete Bins and Silos**

T. A. Meyer
National Silo Association
Louisville, Ky.

**Concrete funnels built from hyperbolic paraboloidal sections provide a**

# Roof for Hunter College Library

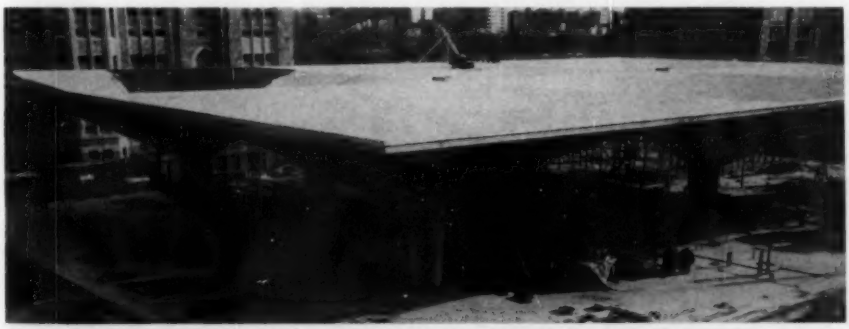
SIX FUNNEL-SHAPED thin concrete shells roof the glass-enclosed library recently completed on the Bronx campus of Hunter College, New York. Four hyperbolic paraboloids are joined to form each of the 60 x 60-ft funnels. Triangular ribs in the valley formed at the juncture of two paraboloids vary in thickness from 8 in. at the top to 14 in. at the bottom. There is a tension rib around the entire outer edge of each funnel.

According to design theory, roof loads are transmitted from the thin concrete membrane to the stiffener ribs by shear, and carried down the ribs by compression to the central supporting column. There is no structural tie between the six shapes, but for purposes of continuity of the library ceiling adjoining slabs of the funnels were key-locked and tied by #3 dowels at the center of shared edges. Underside of the roof is exposed for architectural effect, and a continuous grid containing lighting fixtures hangs 10 ft below the roof's highest point.

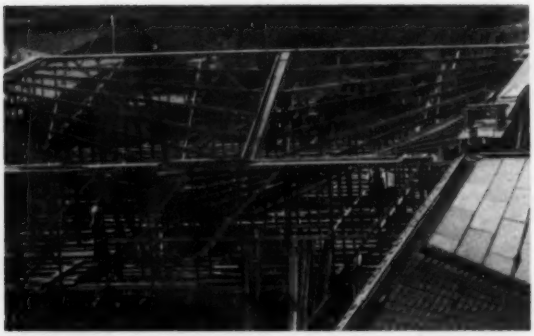
The lightweight shells are 3½ in. thick, the minimum required by the New York City code. High point of each shell is along the outer edge 23 ft above the floor, and the roof slopes to a low point 10 ft above the floor at the columns. Roof slope is as steep as 45 deg near the columns, and averages 36 deg elsewhere.

## Cross-shaped columns

The entire concrete shell, covering a 120 x 180-ft building, is supported on only six columns that taper down from the low point of the funnels, with a roof drain enclosed in each column's center. Resembling a Roman cross in section, the columns are 48½ ft long, and rest on 10 ft square footings placed on soft rock 27 ft below the basement floor and 5 ft below the natural ground water level. Vertical column reinforcement consists of 32 #11 bars and 12 #8 bars; column ties are #4 bars spaced 12 in. on centers. The thickness of the stem of the cross is 14 in.



Hunter College Library as concreting work neared completion. Note Roman cross shape of tapered columns which support the concrete funnels. These columns extend down to 10 x 10-ft footings set on soft rock



Hyperbolic paraboloidal quadrants which make up the funnel-shaped shells are defined by straight lines, so straight timbers set at varying angles served as the basis of these forms. Plywood panels were then warped to the surface outlined by the timbers

## Materials for the shells

The 2-in. slump lightweight concrete was made with an expanded shale aggregate and weighed 108 lb per cu ft. Strengths of 3000 psi were obtained in 7 days and 4000 psi in 28 days, making early form stripping possible. Concrete batch proportions were:

640 lb cement
1140 lb sand
965 lb of ¾-in. lightweight aggregate
5 gal. water per sack of cement
½ lb retarding densifier per sack of cement
air-entraining agent

The shells were reinforced with prefabricated 6 x 6—5-0/5-0 wire mesh supported by reinforcing bars along the compression ribs. Each funnel required 4.2 tons of reinforcing fabric for the membrane, or just over a ton per quadrant. The fabric was delivered in mats 31 ft long, 10½ ft wide, with three mats being placed to a quadrant.

## Construction sequence

Two stage shoring and forms for the shells were erected late in August, 1958, with camber computed to compensate for settling, so that the structure's exterior surfaces would be level after form removal. Since the curved surfaces of the 24 hyperbolic paraboloids, complex as they appear, can actually be defined by straight lines, straight timbers set on variable slopes could be used for form wood. Thin plywood panels, 2 x 8 ft, were easily warped into shape on the straight members to complete the form surface.

The 700-lb reinforcing mats were hoisted by crane and eased into place on 1 in. high chairs attached to the plywood forms. Mats were overlapped at least two wire spacings (1 ft) and nested so that there were only three thicknesses of wire, about 1½ in., at the lap. With the 1-in. chairs below, this afforded at least 1 in. cover for reinforcement, both top and bottom.

To permit nesting at the overlaps, mats had to be placed as nearly parallel with each other as possible, and tied for continuity. Tying began at the center, and the four-man crew worked outward to eliminate and iron out bulging to the edges where excess fabric was trimmed. After the 12 mats were laid and tied for each funnel, the crew placed bars for the stiffener ribs and for the tension rib at the funnel's perimeter.

Concreting began in early September and was completed on schedule in mid-November, with each funnel shell requiring about 60 cu yd of concrete. Parallel sealed gaps left between sheets of the plywood forms to make ribs on the ceiling side of the hyperbolic paraboloid surfaces were grouted by hand prior to concrete placement to eliminate voids. All formwork was wetted down before concreting.

The six self-supporting units were cast separately in an alternating pattern, with work starting on the north side of each shell and proceeding counter-clockwise. Concreting began at the highest point of the funnel and crews worked downward to the low point of each quadrant. Rapid bleeding further reduced the 2-in. slump, enabling the concrete to remain on the steep slope without falling away. The first 60 x 60-ft funnel

*Courtesy Wire Reinforcement Institute*

**First two funnels for library roof have been formed and steel fabric mats are being placed in near shell. Shell at right, with all steel in place, is ready for concreting**

was concreted in about 8 hr, with the fresh concrete of the last strip being vibrated to join the first one placed 8 hr earlier without a cold joint. The succeeding five funnels were concreted in less time as the crews gained skill.

As soon as an area of the concrete roof was rough screeded and rough finished, a chemical curing compound was sprayed on to seal the pores of the fresh mix, prevent rapid evaporation, and insure proper curing. When the concrete had set, shoring of each funnel was removed in concentric rings working inward from the perimeter, so that the load of the shell would be gradually and smoothly transferred.

### Roof treatment

To reduce the cost of heating the library, a 3-in. thickness of perlite insulating concrete was cast directly on the rough screeded roof surface. This layer of material, strength 440 psi, is equal in insulation to more than 30 in. of conventional concrete. Mineral surfaced roll roofing, applied in alternating strips of light and dark grey, adds visual interest to a roof that will be seen from above from a number of taller buildings on the Hunter College campus.

### Construction credits

The library was designed by Marcel Breuer and Associates, architects of New York and The Hague for the Board of Higher Education of New York City. Consulting structural engineers were Farkas and Barron, New York. The general contractor was Leon D. De Matteis and Sons, Inc., Elmont, N. Y. Dic Concrete Corp., also of Elmont, was concrete subcontractor.

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## Omsted elected SEASC president

Harald Omsted, chief structural engineer for the Los Angeles City Board of Education, was recently installed as president of the Structural Engineers Association of Southern California.

Following installation ceremonies, Harry W. Bolin, consulting structural engineer, Ventura, was presented a plaque as honorary member of SEASC.

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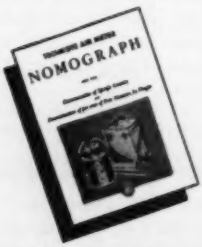


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# Positions and Projects

## Powers addresses ACI Northern California Chapter meeting

T. C. Powers, research counselor, Portland Cement Association, Chicago, currently lecturing to seminar groups at the University of California, was guest speaker at the March 31 meeting of the Northern California Chapter. Among the approximately 60 persons who attended the meeting were ACI vice-presidents Joe W. Kelly and Lewis H. Tuthill.

Mr. Powers, internationally known expert in cement and concrete technology spoke on "The Structural and Engineering Properties of Concrete."

Information regarding future activities of this group is available from the chapter secretary-treasurer, W. E. Moulton, Pacific Cement and Aggregates, Inc., 2625—16th Street, San Francisco 3, Calif.

## New Jersey area moves to form local ACI chapter

The nucleus of another ACI chapter took shape as a group of New Jersey members and associates met on January 29. Approximately 25 members and 20 nonmembers expressed interest in the formation of a local chapter. Spurred by the nonmember interest, several members formed a committee to begin a drive for national memberships in the area.

Information regarding future plans of the group is available from Anthony J. Costanza, structural engineer, 6515 Crescent Boulevard, Pennsauken 5, N. J.

## Michigan ACI members discuss local chapter

A group of nearly 40 ACI members met in Detroit on April 1 to consider forming a local chapter of the Institute. The group decided to delay making a formal request to the Board of Direction but to meet on a regular basis until the extent of interest in such a group can be evaluated.

A committee was selected to plan programs for future meetings within the scope of ACI objectives. The committee includes: William C. Krell, Portland Cement Association; E. A. Finney, Michigan State Highway Department; Elihu Greer, University of Detroit; Warren Yee, Smith, Hinchman and Grylls Associates, Inc.; and Donald Ziegler, Cooper Supply Co.

Chairman for the luncheon meeting was William Krell. E. A. Finney, ACI director, discussed local chapter work with respect to over-all ACI objectives. A movie on the interstate highway program concluded the meeting.

## ASTM committee week held in Pittsburgh

About 1200 persons registered for the annual committee week of the American Society for Testing Materials in Pittsburgh, February 2-6. During the week about 33 committees held nearly 300 committee and subcommittee meetings. New and revised standards and tentative standards that resulted from these meetings will be presented to the society at its annual meeting in Atlantic City in June.

### Mortars for unit masonry

The withdrawal of "Specification for Mortar for Reinforced Brick Masonry" (C 161T) was approved by Committee C-12, Mortars for Unit Masonry. This specification will be replaced by the newer "Specification for Mortar for Unit Masonry" (C 270T). A proposed specification for mortar and grout for reinforced masonry is still under consideration. A new draft will be prepared for consideration at the next meeting of the committee during the annual meeting in June.

Progress was reported on a continuing program of cooperative tests to determine the effect on flow and water retention of a time lag in mixing mortar. These tests are being conducted in cooperation with Committee C-1 on cement.

A new research program will evaluate the influence of aggregate grading on the properties of masonry mortar. Four laboratories will study mortars of low air content and

moderately high air content using a single masonry cement. One sand will be used by all laboratories with at least one additional local sand being used by each laboratory. Five gradings have been selected to cover a range broader than permitted by ASTM limits. Tests will include void content; specific gravity and absorption; flow, as well as flow after suction, and water retention of mortar; unit weight and air content of mortar; Voss plasticity of mortar; and drying shrinkage of hardened mortar.

### Manufactured masonry units

The acceptance of a completely revised specification for clay drain tile (C 4) by ASTM Committee C-15, Manufactured Masonry Units, marks the final phase in the separation of clay and concrete drain tile, both of which were formerly included in Specification C 4. Concrete drain tile is now covered in a separate specification, C 412T.

The study aimed at formulating standard test methods which can be used to evaluate water retardant materials for masonry units continues to present many difficulties. Several task groups are working to isolate the pertinent factors. Progress is being made on a method for determining transpiration; the relation between this property and absorption is also being studied.

The committee is meeting obstacles in its effort to complete a proposed specification for chemical-resistant power packings. Such a specification must include sections covering a number of properties, sizes, shapes, and materials. Round robin tests continue; the latest series will establish effective loading rates.

As a result of industry progress and information now being gathered from standardizing shrinkage tests, extensive revisions of the several specifications for concrete masonry units are in the offing. The committee expects to consider a third draft of proposed revisions at its next meeting. Meanwhile, the present tentative revisions were approved for advancement to standard.

### Fire tests of materials and construction

The "tunnel" test and so-called "small scale" test for determining flame spread were subjects of discussion at the meeting of Committee E-5, Fire Tests of Materials and Construction. The "tunnel" test (E 84T), although recognized by leading code authorities, has been restricted to a limited number of test facilities. This situation has now been relieved by additional facilities and more correlative data are becoming available. Subject to favorable committee letter ballot, this method will be submitted to the society for advancement to standard at the annual meeting. As a separate action, the committee approved a revision of the present tentative to conform with the procedure which is now being used by those conducting this test.

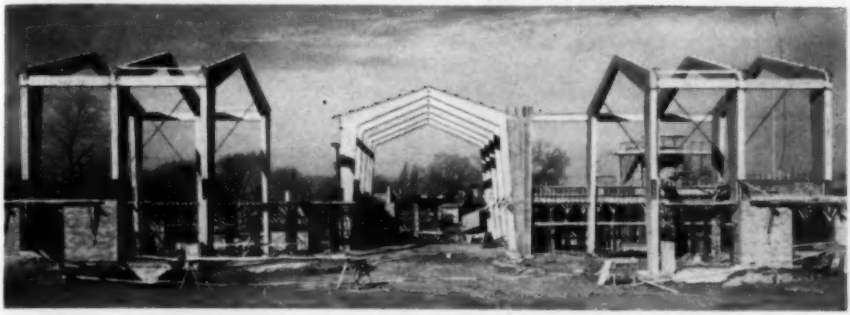
"Methods of Fire Tests of Building Construction and Materials" (E 119), which is the established method referred to in building codes for testing full size panels, will be modified by tentative revisions. These changes pertain to the section on time of testing and, in particular, establish the relative humidity and temperature of air in the seasoning room. A new section on application is to be included which will recognize the use of this test procedure for classification of ceilings which are not an integral part of a floor construction.

Task groups are reviewing at least two of the so-called "small scale" types of fire test. The two receiving the most attention are the radiant panel test and the "small tunnel" test.

### Methods of testing building constructions

Among the things considered by Committee E-6 was the development of methods of testing vapor barriers beneath concrete slabs-on-ground or in crawl spaces. A second draft of a proposed group of test methods which measure the significant properties of vapor barriers is being prepared. A proposed method of test for bond strength of masonry walls, approved at the last meeting of the committee, but not submitted to letter ballot, was reaffirmed and will now be submitted to committee ballot with the addition of certain figures which will aid in the interpretation of the method. A task group has been formed to prepare a method of test of the resistance to shear and diagonal tension of masonry walls.

These soaring two-unit precast concrete arches are 36′ high at mid-point and 42′ wide.



# PRECAST CONCRETE ARCHES form basic structure for new church

Lehigh Early Strength Cement is poured in arch form at Goodstone Mfg. Co. yard. Completed arches were trucked 5 miles to job site.

Three of the four cranes required to position the arches. Note two workmen securing joint between 11½ ton arch halves.

• The new St. Ambrose Roman Catholic Church of Rochester, N. Y. with its eleven precast structural concrete arches, is another example of the growing trend toward precast concrete in modern construction.

The arches, each higher than a 3-story building and weighing 23 tons, were cast in two sections at Goodstone Mfg. Co. plant.

In precasting these arch units, the manufacturer used Lehigh Early Strength Cement for maximum efficiency and economy. For example, forms were stripped and ready for reuse in 18 to 24 hours. And a faster production schedule cut labor costs an estimated 30%.

This is typical of the advantages of Lehigh Early Strength Cement in modern concrete construction.

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*Associate Architect:*
Raymond Ashley, Rochester, N. Y.

*Consulting Engineers:*
Severud-Elstad-Krueger & Assoc., New York, N. Y.

*General Contractor:*
Frank G. Maggio & Bros., Rochester, N. Y.

*Manufacturer of Precast Arches:*
Goodstone Mfg. Co., Rochester, N. Y.

# Building research in Australia

An article by R. E. Banks, head, Building Research Liaison Service, Commonwealth Department of Works, Melbourne, Australia, appeared in the *Journal of the Institution of Engineers, Australia*, December, 1958, describing the development of building research in that country and the organizations which sponsor it. Of interest to those in the concrete field are some brief statements concerning the current work of the Division of Building Research, Commonwealth Scientific and Industrial Organization. The approved program for 1959 includes work on lightweight aggregates, concrete, and concrete floors.

### Lightweight aggregates

Work is being done with expanded clay and shales, as well as an investigation of perlite resources in Australia.

### Calcium silicate hydrate

An earlier study of foam concrete evolved into a study of cellular calcium silicate hydrate, a construction material produced by autoclaving a mixture of hydrated lime and silica flour. It was found to be useful in almost any application where precast concrete would be used and is producible in a wide range of unit weights with high strength to weight ratios developed in a matter of a few hours. A compressive strength of about 6000 psi at the age of 1 day is reported for material weighing 60 lb per cu ft. When prestressed, the loss of prestress was found to compare favorably with the total loss expected in prestressed dense concrete.

### Shrinkage and cracking of concrete

An investigation is being carried out on the cause of cracks in concrete slabs appearing shortly after the concrete is placed and before it is hardened. It was established that these cracks are caused by differential rates of settlement in the components of the concrete when they settle over some obstacles such as reinforcement or a large piece of aggregate. Method of predicting whether or not such cracks will occur is being developed.

### Concrete floors

Investigations of claims that concrete floors on ground are "cold" or "hard" are being made. Instruments were devised and used to measure the instantaneous pressures on the soles of the feet and the deceleration of the head when walking on different types of floor surfaces. These measurements and thermal measurements reveal that the bare concrete floor is not significantly less comfortable than any other bare floor provided the air temperature is the same in each case.

The work was extended to an examination of the effect of concrete floors on the thermal characteristics of dwellings in winter. Calculations were made and checked by measurements in two scale model houses identical

man conquers mountains

And, some day, man will also conquer cancer. With your help.

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except that one had a suspended timber floor and the other a concrete slab on ground. It was found that the differences between the two were comparatively small; the building with the concrete floor had some thermal advantage in the morning and when the outside temperature falls.

### Division of industrial chemistry

In this division work has been done on the setting characteristics of cement, the effects of weathering, corrosion, and dimensional changes on deterioration of mortars in concretes, and other properties of concrete mixes.

It was found that cement paste is surprisingly impermeable if microscopic cracking can be prevented. Shrinkage due to carbonation and other curing phenomena, expansion due to cement-aggregate reaction, the effects of salt solutions, and mechanical stresses are causes of microscopic cracking which are becoming more clearly defined.

The mechanism and general efficiency of air-entraining agents in improving the ease of handling and placing of wet mortar in concrete mixes have been clarified in a long-term project. Further investigation of the effects of agents which improve workability without entraining air has shown that useful results can be obtained without undue expense and without loss of strength in the finished concretes.

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## Alpha Portland predicts increased use of cement

Alpha Portland Cement Co. says in its annual report to stockholders that it has a heavier backlog of contract commitments for 1959 shipments than it has had on its books for many years.

Robert S. Gerstell, president of Alpha, predicts increased use of cement over the coming years due to the expanding knowledge of cement and its applications. He referred to the flexibility of design permitted by structural concrete and the growing use of tilt-up construction, precast and prestressed concrete, soil-cement roads, and the steadily progressing highway program.

The firm announced that it completed installation of a new kiln at its St. Louis plant, making this plant the largest producing unit in the Alpha system. It reported too that it has begun construction of additional silos at the Jamesville, N. Y., plant. These will be capable of storing 100,000 bbl of finished cement, permitting trucks and rail cars to run directly underneath for loading.

## Florida Solite opens plant at Green Cove Springs

Florida Solite, a wholly-owned subsidiary of Southern Lightweight Aggregate Corp., recently opened the first Solite plant in the state at Green Cove Springs, Fla., with W. Glover Shumaker as plant manager.

Sales headquarters are in Jacksonville with George Jones serving as the company's area representative.

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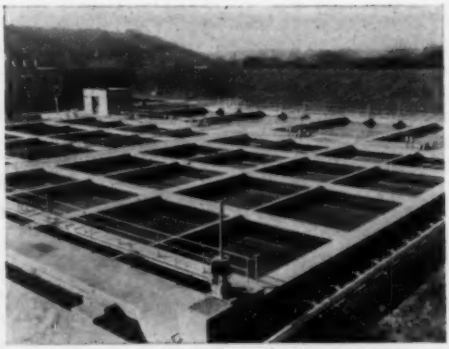
# WATER LEAKAGE PROBLEMS IN CONCRETE STRUCTURES?



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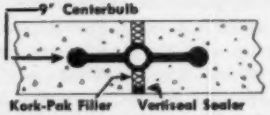


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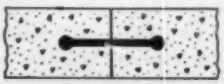

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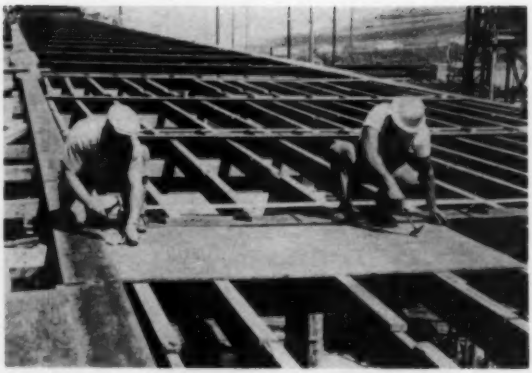


After lightweight steel joists were in place, plywood sheets were nailed to 2 x 4-in. wood nailer strips attached to the top flange of joist. Every component of the falsework was installed manually from the top work level

# BRIDGE DECK FORMING

## uses lightweight structural steel members for temporary support of freshly placed concrete

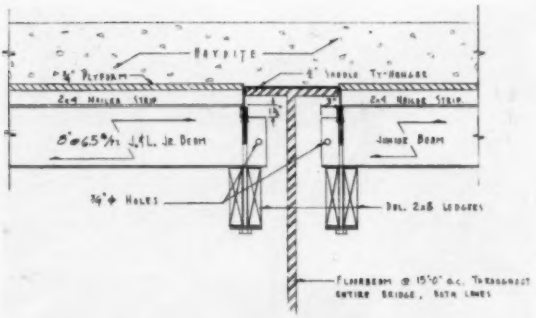
A DECK-FORMING SYSTEM based on lightweight steel I-beams in the falsework supporting the unhardened bridge deck concrete simplified and speeded construction of the 18th Street Viaduct in the new Kansas City Expressway. The light structural members replaced conventional 2 x 8-in. wood joists with an intermediary steel purlin, thus reducing the number of joists from 36 to 13 in most bays.

Two main girders for each double lane of the bridge are spaced 25 to 38 ft on centers, and floor beams are 15 ft on centers across both double lanes for the entire length of the bridge. Hangers 4 ft apart over the structural steel floor beams supported double 2 x 8-in. wood ledgers along both sides of the floor beams (see accompanying detail sketch). The 8-in. I-beams were laid on these ledgers between the floor beams, parallel with the main girders.

These "junior" beams, 14½ ft long, weigh about 100 lb including a 2 x 4-in. wood nailer strip on the top flange. They were spaced 2 ft on centers. Two men could carry them and set them into position from the upper work level of the bridge. Little fabrication was required to prepare the beams for the job. Acetylene torches were used to notch out the top flange on the two ends, and to burn a few holes in the web for handling hooks. Holes were staggered in the top

Seen from below, the simplified falsework of the 18th Street Viaduct in Kansas City, Kan. Workmen space lightweight I-beams 2 ft on centers between main girders. The small beams rest on wood ledgers supported by saddle hangers from the bridge floor beams

*Photographs courtesy Jones and Laughlin Steel Corp.*




Typical deck-forming section at the floor beam

flange for nails that hold the wood nailer strips.

It was unnecessary to fasten the beams to the ledgers or to maintain precise spacing. They were held in position when the standard 4 x 8-ft sheets of ¾-in. plywood were fastened to the nailer strips. The plywood provided the continuous deck surface on which reinforcing steel was laid out before placing the 7¾-in. Haydite concrete road surface.

The roadway was advanced continuously by maintaining work in 32 bays, or 480 ft of each lane of the viaduct, in different stages of development. After the concrete had attained design strength, the lightweight beams were stripped from rear bays and used again in the forward bay forms. Cost and time savings were realized in the stripping as well as the forming operation. The contractor, J. A. Tobin Construction Co., Kansas City, Kan., reported that in one bay six men loosened the ledgers, knocked out and lowered the 17 small I-beams and the 18 pieces of plywood above them, all in 20 min.

The four-lane bridge is a link in the Kansas Turnpike and was built for the Kansas Turnpike Authority. Consultants were Howard, Needles, Tammen and Bergendoff, Kansas City and New York. Robert A. Mandigo was project engineer for the Tobin company.

## Lost mixing time of dual drum pavers

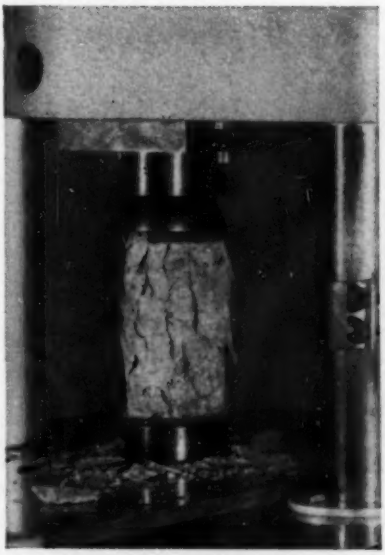
The Bureau of Public Roads, U. S. Department of Commerce, recently produced a new motion picture, *Lost Mixing Time of Dual Drum Pavers*. The film, based on extensive studies of portland cement concrete paving, conducted by the Bureau of Public Roads, highlights the importance of the simultaneous mixing interval in dual drum pavers in meeting mixing time specifications. It shows some trouble spots and the significance of proper adjustments to the batchmeter. Many of the illustrations are animated cutaway views of concrete being mixed within the mixing drums which, followed by live action scenes of field construction operations, make it possible to follow the mixing process as it actually occurs on the job.

This 16-mm sound and color film with a running time of 30 min may be borrowed for showings by any responsible organization by request addressed to Ray B. Dame, chief, Photographic Services, Bureau of Public Roads, Washington 25, D. C. There is no charge except for the express or postage fees.

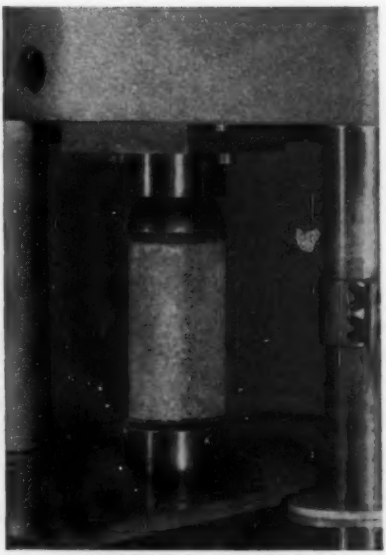
## Richardson elected president of AICE

George S. Richardson, a senior partner in Richardson, Gordon and Associates, consulting engineers of Pittsburgh, has been elected president of the American Institute of Consulting Engineers. Emil H. Praeger of Praeger-Kavanagh, New York, was elected vice-president.

Both Messrs. Richardson and Praeger are long-time ACI members. Mr. Praeger is a member of ACI Committee 216, Fireproofing or Fire Protection of Structures, and Committee 323 (joint ACI-ASCE) Prestressed Reinforced Concrete.



3100 psi Plain Concrete



3850 psi Placewel Concrete

# No extra cost—for today's biggest increase in concrete quality

## Use Johns-Manville liquid Placewel . . . less than 3 fluid oz./sk. of cement

TEST DEMONSTRATES why Johns-Manville Placewel® has been used in over 5,000,000 cu. yds. of concrete. With this liquid admixture, it is possible to use a minimum paste content . . . and get maximum concrete quality.

With Placewel, you use the minimum amount of paste needed to lubricate the mix and bind the aggregate particles together in the hardened state.

By eliminating excess paste you reduce costs, minimize drying shrinkage and make concrete less susceptible to attack by aggressive solutions and weathering.

A Placewel engineer is available on request for technical assistance. Write today for his address. Johns-Manville, Box 14, New York 16, New York.

**Less than 3 fluid oz. of Placewel per sack of cement will—**

- Increase strength up to 30%
- Improve quality at no extra cost
- Increase workability and placeability
- Reduce bleeding and segregation
- Reduce construction, maintenance costs
- Give controlled air-entrainment

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it adds up!

# CLINTON WELDED WIRE FABRIC + concrete = structures built to last

On construction sites across the country, you'll see CF&I-Clinton Welded Wire Fabric. Engineers specify its use because the steel fabric adds the strength of steel to the durability of concrete.

Contractors rely on CF&I-Clinton Welded Wire Fabric to distribute shrinkage stresses evenly to prevent cracking while concrete is setting. It also reduces warping and heaving caused by extreme temperature changes. Should a small crack develop, the steel fabric holds it together so that dirt or moisture cannot expand it.

Made to A.S.T.M. specifications, CF&I-Clinton Welded Wire Fabric is available in a wide range of gages and mesh sizes. Use it on your next job. The nominal cost is repaid many times in reduced maintenance and increased life of the concrete structure. Contact our nearby sales office for complete information.

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THE COLORADO FUEL AND IRON CORPORATION

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*In the East:* WICKWIRE SPENCER STEEL DIVISION—Atlanta · Boston · Buffalo · Chicago · Detroit · New Orleans New York · Philadelphia
CF&I OFFICE IN CANADA: Montreal
6757 CANADIAN REPRESENTATIVES AT: Calgary · Edmonton · Vancouver · Winnipeg

# Prestressing for Texas bowlers . . .

**In the rolling, tree-covered hills midway between Dallas and Fort Worth, this huge black and gray concrete building uses prestressed panels for its wall sections, precast members for the roof, and post-tensioned girders to support the roof members. The million-dollar structure houses 32 bowling lanes, restaurant, sports shop, and other facilities.**

**Prestressed panels measuring up to 24 ft high, 5½ in. thick, are the only wall material. They are brush finished on the outside and have painted interior surfaces. The six girders which span the width of the 98 x 269-ft bowling alley section were formed and cast one at a time on the concrete floor at the base of their supporting columns. Each I-shaped girder is 96 ft long, 80 in. deep, and weighs 64 tons. Ten 1¼-in cables wrapped with creosoted paper were cast in the girder and tensioned when the concrete had hardened. Precast concrete joists 16 in. deep, 6½ ft on centers, span the 39 ft between girders and support the 2 x 6½-ft roof channels. Roof channels were covered with fiberglas over which a built-up roof was applied.**

**The 35,000 sq ft structure was designed by Associated Architects and Planners of Dallas. Great Southwest Corp., Dallas, was general contractor.**

---

## Concrete Pipe Associations announce personnel changes

Howard F. Peckworth, managing director of Concrete Pipe Associations, Inc., Chicago, has announced the promotion of two staff members and the addition of a new staff member.

Marjorie L. Congleton, formerly secretary to the managing director and office manager has been made assistant to the managing director for all three concrete pipe associations: the American Concrete Pipe Association, the American Concrete Pressure Pipe Association; and the American Concrete Agricultural Pipe Association.

John G. Hendrickson, Jr., formerly research engineer American Concrete Pipe Association and American Concrete Pressure Pipe Association has been promoted to director of engineering-research of both associations.

Harold A. Cloud has been appointed assistant managing director of the American Concrete Pressure Pipe Association. He comes to the association following 3 years as research engineer for Minneapolis Honeywell Regulator Co. Prior to that Mr. Cloud was a faculty member of the University of Minnesota agricultural engineering staff.

## Hastings made partner of Ketchum and Konkel

James M. Hastings, who has been an associate in the firm of Ketchum and Konkel, consulting firm of Denver, was recently made a partner. The name of the company will be changed to Ketchum, Konkel and Hastings.

## 1959 CIB congress to be held in Rotterdam

The Congress of the International Council for Building Research, Studies and Documentation (CIB) will be held September 21-25 in Rotterdam and will be open to CIB members, their representatives, and other interested persons. Introductory papers will be read by leading experts and the subjects will be put up for general discussion. The subjects concerned will relate to building research, both fundamental and applied, and to building documentation and transmission of knowledge. Apart from a thorough discussion of the problems presented, the principal object of the congress will be to widen the horizon of the various experts active in the sphere of building.

During the plenary session, simultaneous translation of the proceedings into English, French, and Russian will be provided. Further information is available from the Congress Secretariat, c/o Bouwcentrum, Postbox 299, Rotterdam.

## Engineering Progress Exposition set for New York, June 17-20

The 1959 Engineering Progress Exposition sponsored by the National Society of Professional Engineers and its New York state chapter will be held in New York City from June 17-20 at the Hotel Commodore.

The newest products, processes, techniques, application methods, and services known in modern engineering will be unveiled at the unique show, a major feature of the NSPE silver anniversary convention.

## New mill to double Huron clinker production at Superior

The capacity of the Huron clinker grinding mill at Superior, Wis., will be doubled this summer by the installation of a second compeb mill, according to an announcement by H. R. Schemm, vice-president in charge of operations of Huron Portland Cement Co.

Bringing the total capacity of the plant to 5000 bbl of cement per day, the new mill is scheduled to be in production in July. Contract for the foundation and mill building was awarded to Lakehead Constructors, Inc., of Superior.

## UCLA announces summer short course on prestressed concrete

The Engineering Extension Division, University of California, Los Angeles, announces a summer short course on design of prestressed concrete structures, July 6-17 for persons holding a BS in civil engineering or equivalent.

The purpose of the course is to teach engineers to understand and apply the principles of prestressed concrete for the design of structures. Lectures and laboratory sessions dealing with basic principles of prestressed concrete will cover materials for prestressed concrete; methods of prestressing; loss of prestress; design of prestressed beams and slabs for flexure, shear, bond, and anchorage; camber and deflections; standard members; design of prestressed bridges and buildings; design of members under tension and compression; tanks, columns, and piles; continuous structures; economics; and specifications.

T. Y. Lin, professor of civil engineering, University of California, Berkeley, is course coordinator.

## Dickinson elected to vice-presidency

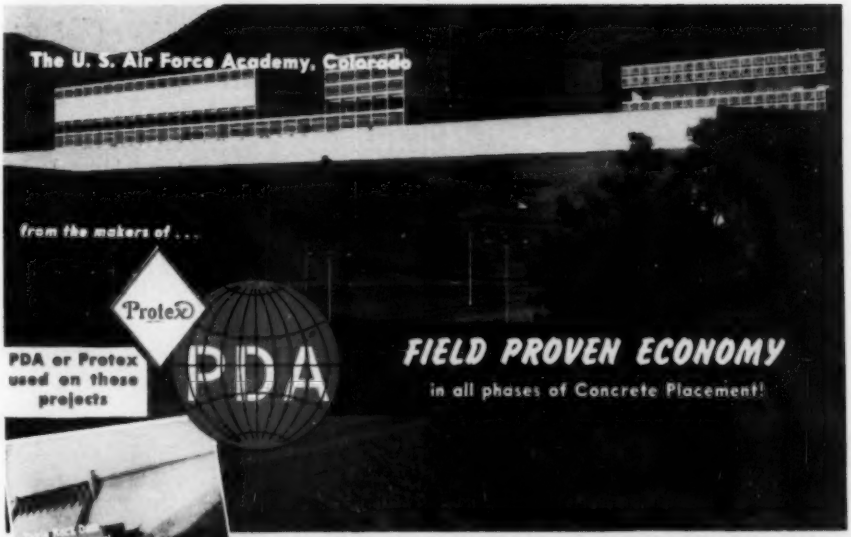
George H. Kimber, has announced the election of William D. Dickinson to the office of vice-president of the Calcium Chloride Institute. Mr. Dickinson has been chief engineer of the institute since 1955 and will continue his responsibilities in that post.

## Electrical problems in cement industry is theme of AIEE meeting

The first American Institute of Electrical Engineers conference wholly devoted to the electrical problems in the cement industry was held at Allentown, Pa., April 14-16, with the Lehigh Valley Section of AIEE cosponsoring the meeting.

Prominent concrete experts participated in the program with C. S. Crawford, president of Whitehall Cement Manufacturing Co., Philadelphia, serving as banquet speaker, April 15.

A similar conference is planned for Milwaukee in November, 1960.

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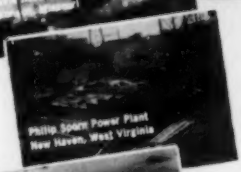
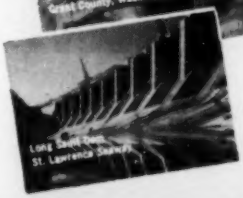
in all phases of Concrete Placement!





**Among the many noteworthy and well-known projects around the world that are using PDA stands the name of the Air Force Academy in Colorado. Joining with this magnificent installation are dams, missile sites, highways, airfields, buildings . . . any construction where the efficient and economical placement of concrete is important!**

### PROTEX Dispersing Agent . . .

**PDA increases strength at any age through water reduction and cement dispersion, improving all the desirable characteristics of concrete, giving a "live" concrete with protection against "hot" weather" slump loss and segregation — allows the placing of controlled durable air entrained concrete without plastic shrinkage cracks. PDA is a selective initial retarder— retarding only the initial set of concrete for extended vibration and finishing time (not delaying form stripping time!) YET it gives no retardation in winter concreting — thus year around benefits are obtained with "all season" PDA.**

### Proven, Dependable, Adaptable

**Proven** in the field . . . PDA insures the successful and economical placement of better quality concrete!

**Dependable** — YES! Proof positive from Government and private projects. Also backed by the world-known and world-respected PROTEX name!

**Adaptable** to any need . . . PDA improves any Pre-stressed, Slip-form, Light-weight, Tunnel-lining, Tilt-up or Lift-slab concrete project!

PDA packaged in a durable bag . . . comes to you in convenient powder form . . . easily mixed and dispensed . . . complete dispensing installations available upon request

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FOR BETTER AND MORE ECONOMICAL PLACEMENT OF CONCRETE, SPECIFY AND USE PDA... FROM THE MAKERS OF PROTEX!

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Industrial & Research Division

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## Past President Moreell awarded "Golden Beaver"

ACI Past President Ben Moreell was among eight men honored this year by The Beavers, honor society of the dam builders and heavy construction experts of the west.

Awarded the "Golden Beaver" under the "special" category, Admiral Moreell was cited as "truly a special man whose career exemplifies the accomplishments and devotion so prominent in leaders of the heavy construction industry."

Admiral Moreell retired as chairman of the board of Jones and Laughlin Steel Corp. in 1958, continuing as director and a member of the executive committee of the board of directors. He joined Jones and Laughlin in 1946 on retiring from active service in the Navy.

Admiral Moreell became a member of ACI in 1930, a member of the board in 1935, and president in 1941. He served 12 years on the Institute's Board of Direction.

## Viest presented ASCE research award

Ivan Viest, bridge research engineer for the AASHO Road Test, Ottawa, Ill., was one of four ASCE members presented research awards at the ASCE Los Angeles convention, February 9-13. He was cited for his research in designing reinforced concrete beams and columns.

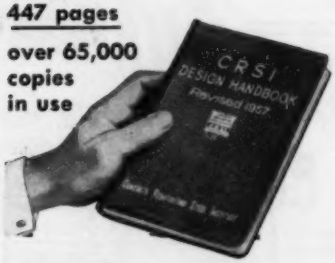
Dr. Viest has been engaged in experimental and theoretical investigations of composite construction since 1948, and during 9 years of association with the University of Illinois played an active part in development of design methods and specifications for composite structures.

An active member of ACI since 1948, Dr. Viest is currently chairman of ACI-ASCE Committee 333, Design and Construction of Composite Structures, and a member of ACI-ASCE Committee 326, Shear and Diagonal Tension.


# The QUICKEST way to get REINFORCED CONCRETE DESIGNS

**Revised 1957 . . . Based on Latest A. C. I. Building Code**

**447 pages**
**over 65,000 copies in use**

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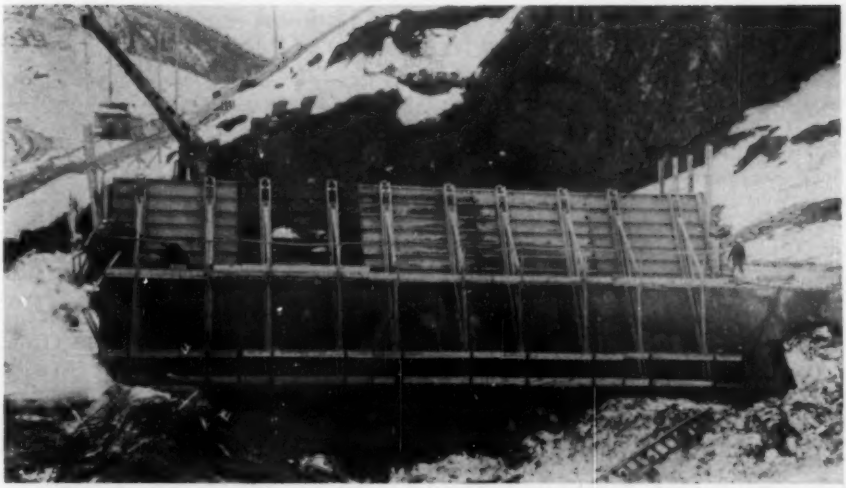
Bring your reference library up to date. Replace your earlier (1952) edition of the CRSI Design Handbook. Completely revised in 1957, this valuable tool will provide you more Reinforced Concrete Designs, all worked out to the latest A.C.I. Building Code. Here is a technical publication of immeasurable value, which the Concrete Reinforcing Steel Institute is making available to architects and engineers for only $6.00 a copy. Send check or money order for your copy, today.

***Prepared by the Committee on Engineering Practice***

**CONCRETE REINFORCING STEEL INSTITUTE**
38 S. Dearborn St. (Div. J), Chicago 3, Illinois



WORKMEN SET FORMS IN PLACE for Monolith No. 9 at Bersimis River Dam now under construction near Forestville, Quebec, Canada. The 7½ ft high forms feature adjustable steel panels for supporting embedded waterstops in the transverse construction joints, and represent an economical departure from the conventional 5 ft high forms. The steel panels, 10 ft wide, are backed by two trussed cantilever uprights. Each upright is supported by a single anchor bolt in the previous concrete lift.

The dam, 280 ft high and 2000 ft wide, will require a million cubic yards of concrete before scheduled completion in 1959. At full production the concreting operation places 165 cu yd per hr, 20 hr per day. Hauled from a nearby batching plant to a 25-ton cableway by an electrically operated transfer car, the concrete is lifted to the body of the dam in 8-cu yd buckets. Although the schedule called for a winter shutdown, much construction has taken place under adverse weather conditions with temperatures ranging from zero to −50 F. For this reason, the steel forms were insulated with a ¾-in. layer of sprayed-on asphalt and asbestos compound designed to last through the construction period of the project.

The Bersimis River Development is directed by Francois Rousseau, chief engineer, Power Development Division, Quebec Hydro-Electric Commission. Perini-Quebec, Inc. is building the dam. Steel forms were supplied by Blaw-Knox Co.

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## 1958 preliminary sand and gravel production report

According to preliminary reports of producers to the Bureau of Mines, United States Department of the Interior, 15 states produced more and 15 states less sand and gravel in 1958 than in 1957. The remaining states produced at about the same rate in 1958 as in the previous year. The national total, approximately 615,000,000 tons, was slightly below the record level of 1957.

## Strandgaard and Bala form civil engineering firm

ACI member Jesper Strandgaard and Stanley M. Bala have announced the formation of the firm of Bala and Strandgaard, civil engineers, with headquarters in San Rafael, Calif.

Mr. Strandgaard was formerly with Sverdrup and Parcel, engineers-architects, San Francisco, and Mr. Bala was with International Engineering Co., Inc., San Francisco.

## Howard M. Zimmerman

Howard M. Zimmerman, chief structural engineer of the Public Buildings Service, Washington, D. C., died March 28.

Mr. Zimmerman received his Bachelor of Engineering degree from John Hopkins University, Baltimore, in 1924 and engaged in design work for the next few years. He was an instructor in structural engineering at John Hopkins University in 1928-29. Mr. Zimmerman entered government service in 1929 as an assistant structural engineer and served actively in this capacity until 1953.

From 1953 to 1957, he was on leave of absence from government service, during which time he served as chief structural engineer for the Washington office of Paul Weidlinger and Associates of New York. During this period Mr. Zimmerman designed such well-known Washington structures as the Arlington Towers, Rittenhouse Apartments, Matomic Building, and others.

In April, 1957, Mr. Zimmerman re-entered government service as chief structural engineer of the Public Buildings Service and continued in this capacity until his death.

In addition to his professional career, Mr. Zimmerman was instrumental in the design of many churches in the Baltimore and Washington area. Many of the churches were designed without fee during his spare time.

Mr. Zimmerman joined the Institute in 1953 and at the time of his death was a member of ACI Committee 315, Detailing Reinforced Concrete Structures.

## Consulting firms merge

Two widely known Wisconsin consulting engineering firms, the Jerry Donohue Engineering Co., Sheboygan, and Robert J. Strass, Inc., Milwaukee, announce the merger of their engineering practices.

The new firm, Hartman-Strass, Inc., consulting engineers, will use the facilities of both companies which have a combined staff of some 30 engineers and draftsmen.

Principals of the firm are Robert J. Strass, president; Bruno J. Hartman, executive vice-president; and associates Carrol R. Maguire, Robert G. Jones, Edward F. Rothe, and William O. White.

## 1958 annual report of the Bureau of Public Roads

During the fiscal year, the greatly expanded long-range highway construction program launched by the federal-aid and federal projects provided improvements on 24,204 miles of roads and streets according to the annual report of the Bureau of Public Roads, fiscal year 1958. Projects for the construction of 35,698 miles of improvements were programed and contracts were awarded for 25,912 miles of road and street improvements. The total federal aid apportioned to the states since the passage of the Federal-Aid Highway Act of 1956 is over $7 billion.

In the fiscal year, 696 miles of pavement were completed on the interstate system, but far more was accomplished in surveys and plans, right-of-way acquisition, and grading and drainage construction. At the end of the year $3.6 billion worth of work was underway or scheduled.

## Martin elected chairman of Michigan highway committee

J. Gardner Martin, district engineer for the Portland Cement Association, Lansing, has been elected chairman of the Michigan State Highway Department Industry Committee. F. Roland Sargent, Saginaw road building contractor, was elected vice-chairman.

The eight-man committee, appointed by Highway Commissioner John C. Mackie to advise him on the problems of the highway construction industry, was created last year.

## Long appointed PCA regional manager

Thomas E. Long has been appointed eastern regional manager of the Portland Cement Association and will supervise PCA field activities in 11 eastern states from the regional office in New York. Mr. Long joined the association in 1947 and has served successively as assistant manager and manager of the farm bureau, and assistant to the vice-president for promotion.

He succeeds Miley J. McMillan, who has been named to the new position of senior consultant in the promotion division.

# Darex Diary

*by Mel Prior*
*Member, American Concrete Institute*
*Technical Service Manager, Construction Products*
*W. R. GRACE & CO.*
*Dewey and Almy Chemical Division*
(No. 13 of a series)

**"Are retarders the same as water reducing agents?"**

THERE seems to be a considerable amount of confusion among the general consumers of admixtures as to the difference, if any, between retarders and water reducing agents. The key word determining the use of one in preference to the other is "temperature."

Water reducing agents are designed to decrease the water/cement ratio of the mix under *normal temperature* conditions, while retarders are expressly designed to delay the initial set of concrete . . . thereby keeping the mix plastic . . . under *high temperature* conditions. Retarders reduce the water/cement ratio of the mix as a side effect.

Under hot weather conditions, the reduction in the water/cement ratio obtained through the use of a retarder is very often reflected in a strength gain in the resultant concrete. Keep in mind, however, that this strength gain results from the retarder delaying the initial set of the mix despite the water-absorbing rays of the hot summer sun. The basic purpose is to combat the harmful effects of heat. Retarders are *not* designed for cold weather concreting.

We recently had one instance where a contractor experienced such outstanding results through the use of a retarder in summertime concreting, that he expressed his intent to continue using a retarder in winter. Since cold weather, of itself, delays the set of concrete, a retarder would have served no useful purpose on the job and could, perhaps, have caused him serious delay. Fortunately, our technical service department was able to point this out to him and furnish him with a water reducing agent . . . the proper admixture for gaining strengths in cold weather concreting.

For hot weather concreting, however, an initial retarder can well be considered essential to maximum efficiency. No matter how high the atmospheric temperature, it keeps concrete plastic until paving is complete by delaying the initial set. Thus, instead of the construction engineer being at the mercy of job temperature conditions, a retarder enables him to control these conditions and produce high quality uniform concrete throughout the job.

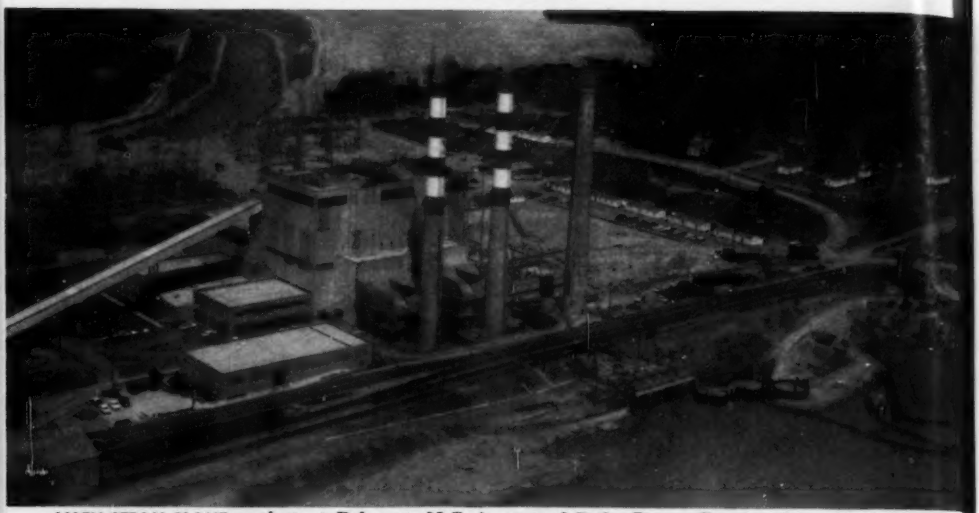
Here at Dewey & Almy, twenty years' experience has gone into the development of an outstanding retarding agent . . . DARATARD. A solution designed to give maximum initial set control. DARATARD is subjected to close chemical control at all stages of the development process from raw material to finished product at our modern concrete physical testing laboratory.

For detailed information on how DARATARD can strengthen your hot weather concreting, write to:

**W.R. GRACE & CO.**
**DEWEY AND ALMY CHEMICAL DIVISION**

**Cambridge 40, Mass.; San Leandro, Calif.; Chicago 38, Ill.; Montreal 32, Canada**

*POZZOLITH . . . makes good concrete better*



**ALLEN STEAM PLANT** project at Belmont, N.C. is part of Duke Power Company's continuing expansion program to meet growing power needs in the Piedmont area. Project includes approximately 73,000 cubic yards of POZZOLITH concrete.

## POZZOLITH concrete specified again for Duke Power Company's continuing $200 million expansion

Duke Power Company—with the largest privately-owned, steam-operated electric capacity in the southeast—handles its own design, engineering, construction and concrete production. These operations are directed by C. T. Wanzer, Chief Engineer —C. E. Watkins, Construction Manager and R. L. Dick, Resident Engineer.

On the Allen Steam Plant project—several concrete mixes were designed with POZZOLITH and Duke Power Company's fly ash. The mix that best met job requirements most economically was:

Cement— 353 lbs.
Fly Ash (Duke Power)— 117 lbs.
Sand—1215 lbs.
1" Crushed Stone—2057 lbs.
Total Water— 30 gal.
POZZOLITH

The job concrete was easily pumped more than 600 feet through an 8-inch line from the automatic batching plant. Slump of 2" to 3" was maintained—there was excellent workability and cohesiveness . . . and high, uniform strength . . . 2250 psi at 7 days . . . 3700 psi at 28 days . . . and 4900 psi at 90 days. The specified 28-day strength of 3000 psi was easily exceeded.

The local Master Builders field man will welcome discussing your requirements. Remember, POZZOLITH concrete is higher in quality and more economical than plain concrete or concrete made with any other admixture.

*The Master Builders Company, Cleveland 3, Ohio*
*Division of American-Marietta Company*
*The Master Builders Co., Ltd., Toronto 9, Ont.*
*International Sales Dept., New York 17, N. Y.*
*Branch Offices in all principal cities.*

# MASTER BUILDERS® POZZOLITH*

**POZZOLITH is a registered trademark of The Master Builders Company for its concrete admixture to reduce water and control entrainment of air and rate of hardening.*

## Turner announces firm name change

N. P. Turner, managing partner of Freese, Nichols and Turner, consulting engineers, Houston, Tex., announces a firm name change to Freese, Nichols, Turner and Collie.

## Construction started on Los Angeles Hall of Records

Twaits-Wittenberg Co. and Morrison-Knudsen Co., Inc., joint venture constructors and engineers, have started construction of the $11½ million county of Los Angeles new Hall of Records.

The structure, which will be the tallest of the new Los Angeles civic center buildings, will consist of eight floors plus basement and penthouse in the main part and 15 mezzanine type floors in the stack area, and will enclose more than 460,000 sq ft. In addition to structural steel and cast-in-place concrete construction, the building will feature exteriors of ceramic veneer and granite.

## McFarland and Johnson form consulting firm

William H. McFarland and John W. Johnson have established the firm of McFarland-Johnson, consulting engineers, with offices in Binghamton and Buffalo, N. Y.

Mr. Johnson was formerly New York State superintendent of public works.

## Petzar transferred to Phoenix as PCA district engineer

The Portland Cement Association has announced the opening of a new district office in Phoenix, Ariz., with George M. Petzar, formerly of the Los Angeles office, as district engineer.

Mr. Petzar joined PCA in 1954 as a general field engineer, working in concrete and soil-cement paving, conservation, and other fields of cement use. Since 1956 he has specialized in technical service in the housing field.

Gene R. Morris, general field engineer in the Tucson area, will continue in that capacity. A second field engineer will be added to the Phoenix office staff.

## Selden appointed ABPA technical director

John K. Selden, Toledo, has been appointed technical director of the Autoclave Building Products Association.

Mr. Selden has had more than 20 years experience in autoclaving and is the author of many papers on that subject. He will visit plants employing autoclaving in the curing of their products and will also advise producers on modern methods and equipment. Mr. Selden is a member of ACI Committee 716, High-Pressure Steam Curing.

## LOOKING AHEAD

May 13-15, 1959—"Deutsche Betontag 1959," German Concrete Association, Congress Hall of the German Museum, Munich, Germany

May 18-19, 1959—39th Annual Meeting, The Society of American Military Engineers, Washington, D. C.

May 25-26, 1959—Wire Reinforcement Institute, The Greenbrier, White Sulfur Springs, W. Va.

May 28-30, 1959—Concrete Reinforcing Steel Institute, The Greenbrier, White Sulphur Springs, W. Va.

June 21-26, 1959—Annual Meeting American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

Aug. 10-12, 1959—Conference of Lightweight Concrete Block Manufacturers, National Cinder Concrete Products Association, Chalfonte-Haddon Hall, Atlantic City, N. J.

**Nov. 3-5, 1959—Regional Meeting, American Concrete Institute, Hotel Continental Hilton, Mexico City, Mexico**

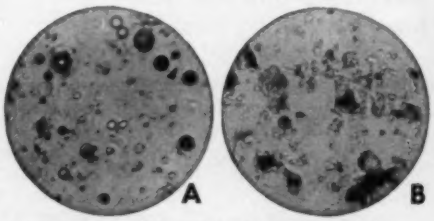
# How much CEMENT... How much FLY ASH... How much AGGREGATE ...go into the best mix for a given requirement ?

**THAT** correct design of the mix is vital is not open to question. The best results are obtained when ingredients are proportioned in accordance with information obtained from extensive investigation. Programs of this kind are continuing. The information developed is available through the companies named below.

## Fly Ash Improves Quality in Several Ways

A great many tests have shown that a concrete containing proper amounts of an approved fly ash is superior to a concrete made without fly ash . . . Fly ash concrete is easier to place and finish, and has a lower heat of hydration. It also has greater resistance to the penetration of water and to the effects of aggressive waters.

All these are important, depending on the purpose of the concrete. One quality *always* desired is high workability, ease of placing in the forms, filling them completely, using even less water, with a consequently denser mix and more watertight job. Main reason for the high workability is seen in microscopic views A and B. The particles of cement (B) are rough, angular, while those of fly ash (A) are mostly spherical and smooth. Thus the "ball bearing" effect which makes the mix handle more easily.




*Each of the Companies Below Has Technical Data and Competent Engineers to Help You in Designing the Most Effective Mixes Employing Cement and Fly Ash*

**CHICAGO FLY ASH COMPANY**
**228 N. La Salle St., Chicago 1, Ill.**

**WALTER N. HANDY COMPANY, INC.**
**P. O. Box 549, Evanston, Ill.**

**McNEIL BROTHERS, INC.**
**P.O. Box 4015, Bridgeport 7, Conn.**

**DETROIT EDISON COMPANY**
**2000 Second Ave., Detroit 26, Mich.**

# Honor Roll

January 1—April 30, 1959

ACI membership is essential to professional advancement in the concrete field since it offers up-to-the-minute "know-how" on problems encountered.

The membership committee recommended a new plan for crediting individuals for obtaining new members as follows: 1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing. This new system has been made retroactive from Mar. 1, 1959.

Heading the Honor Roll this month is Samuel Hobbs with 30 credits. Robert P. Witt is second with 25½ credits.

| | |
|---|---|
| Samuel Hobbs | 30 |
| Robert P. Witt | 25½ |
| Chas. W. Cole, Jr. | 9 |
| Kenneth M. Huber | 9 |
| A. T. Klassen | 9 |
| David A. Sauer | 8 |
| Henry Aaron | 6 |
| John L. Burdick | 6 |
| S. D. Burks | 6 |
| Mauro N. Guzman | 6 |
| John E. Heer, Jr. | 6 |
| George J. Kerekes | 6 |
| W. S. Cottingham | 5 |
| Jack C. McCoe | 5 |
| Gene M. Nordby | 5 |
| George B. Southworth | 5 |
| Ferruh Taskin | 5 |
| Michael Alexander | 4 |
| James Chinn | 4 |
| George P. Duecy | 4 |
| Gregorio Hernandez | 4 |
| B. E. Kester | 4 |
| Charles C. Luther | 4 |
| H. C. Pfannkuche | 4 |
| Joseph J. Shideler | 4 |
| Glenn C. Thomas | 4 |
| Lewis H. Tuthill | 4 |
| Robert F. Adams | 3 |
| Tamnoon Ansusinha | 3 |
| John D. Antrim | 3 |
| Edward E. Baker | 3 |
| D. E. Bowen | 3 |
| Jose Maria Bravo | 3 |
| Ralph B. Brenan, Jr. | 3 |
| C. F. Brown, Jr. | 3 |
| Theodor H. Busck | 3 |
| Guillermo Castellanos G. | 3 |
| Ralph G. Crimms, Jr. | 3 |
| Frederick W. Drury, Jr. | 3 |
| James Lee Ford | 3 |
| Ray J. Foss | 3 |
| Junius R. Gardner | 3 |
| Ben C. Gerwick, Jr. | 3 |
| Martin J. Gutzwiller | 3 |
| Robert B. Harris | 3 |
| Eric J. Hayford | 3 |
| Horace G. Hill III | 3 |
| Aleck E. Hiscox | 3 |
| Samuel I. Iwata | 3 |
| J. E. Jones | 3 |
| Elias Kardaras | 3 |
| William W. Karl | 3 |
| Joe W. Kelly | 3 |
| Frank Kerekes | 3 |
| Frank R. Killinger | 3 |
| Edward Laing | 3 |
| Ronald Lazar | 3 |
| Elmer C. Lee | 3 |
| Janis Liepins | 3 |
| Henry Lipkind | 3 |
| Russ A. Loveland | 3 |
| Douglas McHenry | 3 |
| Ernst Maag | 3 |
| M. F. Macnaughton | 3 |
| Frank B. May | 3 |

**Looking ahead to**

## *1960 in New York City*

ACI's Technical Activities Committee is already shaping up program plans for the 1960 convention to be held in New York City, March 14-17.

Those who intend to offer papers for presentation at the 56th annual meeting should write to Institute headquarters before July 1, furnishing a synopsis which should make clear the scope of the proposed paper and indicate features that the author thinks will justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1959, and final manuscripts of accepted papers will be due Jan. 1, 1959.

From the replies received and suggestions from other sources, TAC will select papers to make up each convention session.

Write Institute headquarters NOW about the papers YOU want to see on the 1960 program

William J. Moore ..... 3
Newlin D. Morgan, Jr. ..... 3
Donald H. Olson ..... 3
Rafael Enrique Pacheco ..... 3
Jack L. Randall ..... 3
Jerome M. Raphael ..... 3
Thomas J. Reading ..... 3
Raymond C. Reese ..... 3
Antonio I. Rosquete ..... 3
Paul Serfass ..... 3
L. Shector ..... 3
C. P. Siess ..... 3
Howard Simpson ..... 3
Stuart H. Snyder ..... 3
Ralph W. Spencer ..... 3
Willard Stevenson ..... 3
T. Tassios ..... 3
F. D. Taylor ..... 3
Pastor B. Tenchavez ..... 3
J. Antonio Thomen ..... 3
John P. Thompson ..... 3
Sophus Thompson ..... 3
Ellis S. Vieser ..... 3
Bernardo Villegas ..... 3
Stanley G. Zynda ..... 3
Pedro M. Bassim ..... 2
Walter E. Blessey ..... 2
John E. Breen ..... 2
Boris Bresler ..... 2
Paul E. Chen ..... 2
P. Q. Chin ..... 2
Phil M. Ferguson ..... 2
Ambrosio R. Flores ..... 2
George E. Large ..... 2
Leo M. Legatski ..... 2
Joachim F. Leppmann ..... 2
Leo Liberthson ..... 2
Ernest C. Marmet ..... 2
Wendell H. Nedderman ..... 2
Donald A. Sawyer ..... 2
A. M. Tovar ..... 2
Jose R. Perez Valentin ..... 2
C. K. Wang ..... 2
J. H. Weaver ..... 2
Roberto Zepeda Aldana ..... 2

# New Members

**The Board of Direction approved applications in the following catagories: 67 Individual, 3 Corporation, 15 Junior, and 22 Student—making a total of 107 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership on Apr. 1, 1959, was 9866.**

### Individual

ABASTILLAS, BEN, Manila, Philippines (Assoc. C. E., Irrigation Div., Bur. of Pub. Wks.)
ADJELEIAN, JOHN, Ottawa, Canada (Cons. Struct. Engr., Adjeleian, Goodkey, Weedmark & Assoc., Ltd.)
ANHALT, LEONARD P., Milwaukee, Wis. (Struct. Engr., Lefebvre, Wiggins & Assoc.)
ARNOLD, ROBERT S., Fawkner, Australia (Sales Mgr., Constr. Chem. Prods., Dewey & Almy Pty, Ltd.)
AUFMUTH, R. B., Cleveland, Ohio (Chf. Engr., H. K. Ferguson Co.)
BANISTER, BILLY N., Corpus Christi, Tex. (Gen. Field Engr., PCA)
BERGHINZ, CARLO, Milan, Italy (Mgr., Overall Operation & Tech. Dept., Electroconsult S. p. A.)
BIRKMYER, JAMES, Toronto, Canada (Mgr., W. S. Atkins & Assocs.)
BORCHGREVINK, THEODOR, Stepney, Conn. (Cost Engr., Designer, Constr. Engr., Frouge Constr. Co., Inc.)
BROSSET, NOEL M., Paris, France (France Benelux Mgr., The Master Bldrs., Int. Sales Dept.)
BUGGY, CHARLES E., Los Angeles, Calif. (Constr. Est., Alta-Fraser-Edwards Co.)
COOTE, CHARLES W., Princeton, N. J. (Struct. Engr., Bur. of Yds. & Docks, U. S. Navy)
COVELLO, THOMAS, Edmonton, Canada (Assoc. Engr., Diamond-Clarke & Assoc.)
CUMMINS, KENNETH DONALD, Detroit, Mich. (Assoc. Prof. of Civil Engr., Univ. of Det.)
DAVIDSON, J. KNOX, Montreal, Canada (Chf. Engr., Louis Donolo, Inc.)
DAVIS, FRANK D., Los Angeles, Calif. (Cement Colors)
DE ARELLANO, GERMAN RAMIREZ, Santurce, Puerto Rico (Arch.)
FELDSTEIN, JOSEPH, Los Angeles, Calif. (Struct. Engr., Inter-Continent Engrg. Co.)
GABRIEL, NORRIS R., Memphis, Tenn. (Struct. Engr.)
GEIL, GEORGE H., Camp Hill, Pa. (Struct. Engr. & Insp., Bourquard, Geil & Assocs.)
GIANOTTI, EDEN J. J., Mt. Vernon, N. Y. (Matls. Engr., Morrison, Knudsen, Oman)
GILL, RICHARD FRANK, London, England (Sr. Designer, Draftsman, Stone & Webster Engrg. Co., Ltd.)
GILVEY, EDWARD E., Pottstown, Pa. (Struct. Engr., Supv., Struct. Engrg. Dept., Sanders & Thomas Assocs.)
GREENFIELD, PAUL N., Los Angeles, Calif. (Struct. Engr.)
GULBINOWICZ, HENRY, Platteville, Wis. (Asst. Prof. of Civil Engrg., Wis. Inst. of Tech.)
HARDING, MALCOLM E., San Pedro, Calif. (Sales Conc. Specialties & Prof. Heavy Constr. Est.)
HENNING, CURTIS R., Redwood City, Calif. (Pres., Concrete Chemicals Corp.)
HUSAIN, M. M., Karachi, Pakistan (Exec. Engr., Ministry of Food, Gov't of Pakistan)
JAKOBSON, ROBERT W., Minneapolis, Minn. (C. E., Thorshov & Cerny, Inc.)
KAY, ALAN A., Vancouver, Canada (Chf. Engr., G. S. Eldridge & Co., Ltd.)
KESSMEIER, ERIK G., Worthington, Ohio (Distr. Repres., Field Service, The Master Bldrs. Co.)
KEYES, CYRIL C., Vancouver, Canada (Pres., Gunite & Grout Co.)
KHOURY, E. M., Canoga Park, Calif. (Cons. Engr., Civil & Struct.)
KNIGHT, KENNETH K., Great Falls, Mont. (Arch. & Engr.)
KOFFLER, DANIEL, New Castle, Del. (Daniel Koffler & Assocs., Cons. Engrs.)
KRAPFENBAUER, ROBERT J., Vienna, Austria (Cons. Engr., Zivilingenieurkammer fur Wien)
KREIGH, JAMES D., Tucson, Arizona (Asst. Prof., Dept. of Civil Engrg., Univ. of Arizona)
LAU, CHRISTOPHER Y., Oakland, Calif. (Supv. Struct. Engr., Div. of Arch., State of Calif.)
LEGLER, TED R., East Chicago, Ind. (Supt. of Constr., Youngstown Sheet & Tube Co.)
LENKO, GEORGE, New York, N. Y. (Supv. Matls. Engr., U. S. Army Corps of Engrs.)
LEON, ENRIQUE ROBLES, Guadalajara, Mexico (C. E.)
MALIN, LEONARD E., East Chicago, Ind. (Chf. Engr., Western Div., Youngstown Sheet & Tube Co.)
MCHENRY, CHARLES STEWART, Dearborn, Mich. (Aggregate Surfaces, Inc.)
MCLAUGHLIN, ROBERT R., Sacramento, Calif. (Chf. Engr., Delta Prestress Concrete)
MONTOURI, CESARE, Napoli, Italy (Civil Struct. Engr.)
MORMAN, FRANK C., Monrovia, Calif. (Tech. Sales Repres., Marathon Div. of American Can Co., Chem. Sales Dept.)

Mosher, L. W., Detroit, Mich. (Cons. C. E.)
Mullens, Glenn B., Laramie, Wyo. (Prof. of Civil Engrg., Univ. of Wyo.)
Near, J. D., Toronto, Canada (Dir. of Operations Div., Dept. of Pub. Wks., City of Toronto)
Ng, Lin-Fai, Los Banos, Calif. (Design of R/C Structures, Hydraulic Design of Irrigation System, Stoddard & Karrer, Civil Engrs.)
Popovics, Sandor, W. Lafayette, Ind. (Grad. Asst., Purdue Univ.)
Rickerl, Jerome B., Omaha, Neb. (Maintain Field Control of concrete construction, U. S. Corps of Engrs.)
Romig, John R., Colton, Calif. (Chf. Chem. & Chem. Engr., California Portland Cement Co.)
Rowland, Robert L., Wisconsin Rapids, Wis. (Arch., Rowland Assocs.)
Smith, H. Peter, Toronto, Canada (Sr. Matls. Engr., Matls. & Research Section, Dept. of Hwys., Ontario)
Solnok, Albert C., Tulsa, Okla. (Partner, Netherton-Dollmeyer-Solnok, Cons. Engrs.)
Spamer, Alfred B., Baltimore, Md. (Field Engr., PCA)
Spiegel, Herman D. J., New Haven, Conn. (Asst. Prof. of Arch. Engrg., Yale Univ.)
Stinchfield, William E., Bridgeport, Wash. (Resident Engr., Chf. Joseph Dam, Corps of Engrs., U. S. Army)
Stout, John B., Jr., Pittsburgh, Pa. (Sr. Proj. Engr., Pittsburgh Coke & Chemical Co.)
Switzer, Eugene L., Corona, Calif. (Matls. Engr., A & E Labs.)
Toyomura, Dennis T., Chicago, Ill. (Arch.)
Trischank, August, East Chicago, Ind. (Engr., Youngstown Sheet & Tube Co.)
Vogt, Ernest L., Jr., Houston, Tex. (Cons. Struct. Engrs.)
Walker, C. W., Chicago, Ill. (Vice-Pres., Wyzenbeek & Staff, Inc.)
Weber, Neal E., Los Angeles, Calif. (C. E., Southern Calif. Edison Co.)
Wong, James Poy, San Francisco, Calif. (Sr. Struct. Designer, Div. of Arch., State of Calif.)

### Corporation

Advance Cast Stone Co., Milwaukee, Wis. (Bruce Garni, Pres.)
Goldblatt Tool Co., Kansas City, Mo. (Melvin L. Goldblatt, Vice-Pres.)
Raymond G. Osborne Labs., Inc., Los Angeles, Calif. (Raymond G. Osborne, Pres.)

### Junior

Alagia, Jayantilal Shamjibhai, St. Louis, Mo. (Struct. Engr., Sverdrup & Parcel, Inc.)
Andreu, Antonio, Minneapolis, Minn. (Stud., Univ. of Minn.)
Apeland, Kristoffer, Berkeley, Calif. (Res. Scientist C. E., Univ. of Calif., Engrg. Matls. Lab.)
Buckmaster, B. Gail, Tucson, Ariz. (Arch., Nicholas Sakellar, Arch.)
Flannery, Curtis E., Urbana, Ill. (Proj. Dir., Eberhard & Murphy, Archs.)
Prud'Homme, O. Guy, L'Assomption, Canada (Job Insp., Prud'Homme & Freres Ltd.)
Lee, Edgar K. M., Berkeley, Calif. (Engr. in Trng, Chin & Hensolt, Struct. Engrs.)
Leslie, Gordon, Edmonton, Canada (Sessional Instructor, Univ. of Alta.)
McDonald, Robert B., Columbus, Ohio (Lab. Tech., Marble Cliff Quarries Co.)
Prentzas, Elias G., Davenport, Iowa (Plant Engr., Central Pipe & Prestress Corp.)
Rivas-Roman, Ricardo, Caracas, Venezuela (Pres., Constructora Elcosa)
Rivlin, Ziev J., New York, N. Y. (Sales Engr., L. Sonneborn Sons, Inc.)
Roa, Everaldo Antonio, Ciudad Trujillo, Rep. Dom. Prof. of Engrg., Santo Domingo's Univ.)
Shields, Thomas E., New York, N. Y. (C. E., Hwy Dept., Howard, Needles, Tammen & Bergendoff)
Uzcategui B., Rodoscal, Caracas, Venezuela (C. E., Municipal Council of Caracas)

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**Student**

AMADOR, JOSE GABRIEL, Quebradillas, Puerto Rico (Univ. of Puerto Rico)
AMAYA DE LEON, Rolando, San Salvador, El Salvador
COSTA, AGUSTIN R., Minneapolis, Minn. (Univ. of Minn.)
CROOKS, ROBERT W., W. Lafayette, Ind. (Purdue Univ.)
DROEMER, WILLIAM WALTER, Bryan, Tex. (A & M College of Tex.)
ESQUENAZI, CH., EDMUNDO, Boston, Mass. (MIT)
ESTEVA MARABOTO, LUIS, Boston, Mass. (MIT)
GAVIN, JOHN O., New Orleans, La. (Tulane Univ.)
HELLSTROM, RICHARD B., Gainesville, Fla. (Univ. of Fla.)
HERNANDEZ, DENNIS W., Santurce, Puerto Rico (Univ. de Puerto Rico)
JINDAL, ROOP LALL, Champaign, Ill. (Univ. of Ill.)
KROKOSKY, EDWARD M., Cambridge, Mass. (MIT)
LINZELL, LEWIS EDWIN, Columbus, Ohio (Ohio State Univ.)
MARTIN, C. CLIFFORD, JR., Richardson, Tex. (A & M College of Tex.)
MARTINEZ MONRO, RAFAEL, Caracas, Venezuela (Univ. de Catolica)
NICKERSON, DONALD W., Billerica, Mass. (Tufts Univ.)
OUELLETTE, PAUL L., North Cambridge, Mass. (Tufts Univ.)
PARKER, JOHN E., Columbus, Ohio (Ohio State Univ.)
PINCUS, GEORGE, Decatur, Ga. (Ga. Inst. of Tech.)
SANCHEZ, H. M., Kingston, Canada (Queen's Univ.)
SHA, CHANDRAKANT D., Ann Arbor, Mich. (Univ. of Mich.)
WITZKE, EDUARDO F., Caracas; Venezuela (Univ. de Catolica)

## Tools, Materials, Services

**Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.**

### Rejuvenate traffic-worn concrete

Emeri-Crete Kure, liquid sodium silicate compound, offers a quick and economical cure to bothersome dusting conditions and is a means of rejuvenating old, traffic-worn concrete according to the manufacturer. Special penetrating properties seal and harden the surface against weather and traffic by converting the old cement's crumbling "free lime" into solid calcium silicate. Available in 1, 5, and 50 gal. containers, this compound can be applied with hand or power spray, hair broom, or burlap drag.—Walter Maguire Co., Inc., 60 East 42nd St., New York, N. Y.

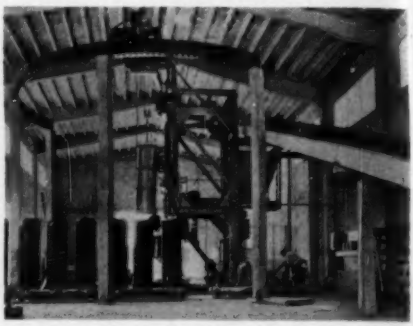
### Prestressed concrete anchor assemblies

Cleveland Forge Works announce a post-tensioned, prestressed concrete wire anchor assembly. The drop forged, heat treated, and magnafluxed unit is designed for use with the BBRV system.

The assembly will anchor 12 wires, ¼-in. diameter, after each wire has been individually headed on each end. Spacer shims of 3/16, ¼, and 5/16 in. are supplied in addition to ¾-in. locking shims for tension adjusting—H. K. Porter Co., Cleveland Forge Works, 3270 East 79th St., Cleveland 4, Ohio

### Circular crane and hoist service concrete pipe placing operations

A 6-ton capacity circular crane and hoist is now in operation at the Fountain Sand and Gravel Co., Pueblo, Colo., to aid in placing concrete for reinforced



concrete pipe. The unit was manufactured by the Wright Hoist Division, American Chain and Cable Co., Inc., York, Pa.

Photo shows the arrangement for the hoist which is suspended from a semicircular track and closely mounted to the ceiling of the production area. The hoist unit, operating from a power supply source of 240 volts, 60 cycle, 3-phase current, is of ample capacity to handle any size of concrete pipe manufactured by Fountain Sand. A conveyor belt, to the right, is used to supply the bin hopper with sand, gravel, and cement. Water is furnished through a system of proportioning valves.—American Chain and Cable Co., Inc., 929 Connecticut Ave., Bridgeport 2, Conn.

### Power buggy

Aeroil Products has announced the introduction of their Model PL-10 power buggy with bucket capacity of 10 cu ft. Powered by a 6-hp engine, the unit will travel at maximum speed of 5 mph up a 20 percent slope with 1500-lb payload according to manufacturer.

Clutch and brake are operated from one hand lever while throttle control is on other hand lever; forward and reverse gears operated by foot pedal. Flat pallet is available in place of the bucket for hauling bricks, blocks, boxes, etc.—Aeroil Products Co., Inc., 17 Wesley St., South Hackensack, N. J.

**Nonstaining sealant**

DEL synthetic rubber compound Special Gray J-31 is specifically designed for use on white marble and other light-faced stone and masonry joints. When used with a special primer, DEL Primer J-32, this sealant will not stain the most delicately colored marble or stone according to the developer.

Company states that DEL is easy to apply and adheres to most materials, including the "curtain wall metals." Further state that it "gives" without cracking under stress and strain of normal building movement and joints will not open due to the expansion and contraction resulting from temperature fluctuations. —David E. Long Corp., 220 East 42nd St., New York 17, N. Y.

**Lightweight slag aggregate**

Process developed by the Dow Chemical Co. converts molten slag from wet bottom, slag tap boilers from a waste material to lightweight aggregate. Aggregate produced by this process by Dow has been trademarked Dowlite. It is an iron aluminum silicate that exhibits both light weight and strength characteristics as well as good free-flow qualities in block machines according to the manufacturer.

Company spokesman further states that Dowlite has the appearance of black glass when cold and is designated as amorphous. The aggregate is inert and contains no unburned combustible materials thus preventing "popouts" which can occur in block made from aggregates containing unburned refuse. Information on the new Dow process is available.—H. L. McNally, LWA Project Engineer, The Dow Chemical Co., Midland, Mich.

**Form coating**

Brad Chemical recently introduced Poly-Kote FG-54, a concentrated form coating. One gallon of Poly-Kote FG-54 is added to 54 gal. any grade fuel oil.

Poly-Kote FG-54 is effective on new and used forms of wood, metal, or plastic, and can be used with any type of concrete mix. As it dries it forms a flexible surface impervious to water and the chemical action of drying concrete according to manufacturer. Nonacidic and noncorrosive to equipment; no color transference to the finished concrete face nor is there any oily deposit on stripped forms.—Brad Chemical, Inc., 111 W. Washington St., Chicago, Ill.

**Simplified foundation slashes cost of NAHB research house**

In constructing a three-bedroom research house at Knoxville, Tenn., National Association of Home Builders Research Institute chose to use simplified techniques and economical materials to produce a house that was well within reach of low-income groups with no sacrifice in quality.

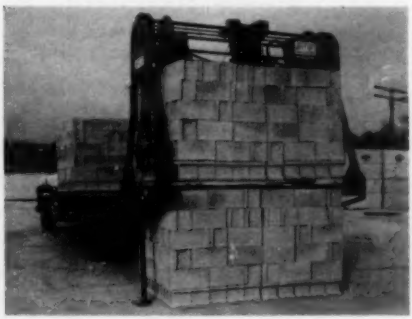
The home's foundation features a floor slab of Betocel insulating foam concrete. The product, manufactured by Reflectal Corp., Chicago, architectural subsidiary of Borg-Warner Corp., consists of normal mortar sand, normal cement, water, and the special Betocel bubble-forming emulsion, mixed on the job.

According to the manufacturer a major advantage of this form of construction is that the foam concrete itself serves as both slab and edge insulation, thus eliminating labor cost involved in installing a separate edge insulation. Density can be easily varied from 20 to 75 lb per cu ft by proportioning the ingredients in the Betocel mixer. Manufacturer further states that Betocel is fireproof and fungus proof, and highly resistant to cold and heat.—Reflectal Corp., Chicago, Ill.

**Block loader**

The 1959 Jiffy-Lift one-man operation loader and unloader handles 10,000 lb with ample depth of drop for all conditions according to the manufacturer.

Designed for loading or unloading concrete block; brick, either palleted or banded; stone, tile and pipe or



precast concrete, the unit features four-wheel drive and four-cable suspension. Brochure covering uses and specifications available.—Elberfeld Manufacturing Co., Inc., Elberfeld, Ind.

**Concrete grinding attachment**

Manufacturers of Wyco concrete vibrators announce an attachment for their three sizes of junior vibrators for wet or dry grinding of concrete. By removing the vibrator head and attaching the new angle head, the machine is changed from a vibrator to a concrete grinder. A speed reducer is built into the tool which greatly increases the torque and gives the proper grinding speed to the cup-shaped grinding wheel. Entire unit including vibrator motor weighs 20 lb. —Wyzenbeek and Staff, Inc., 223 N. California Ave., Chicago 12, Ill.

**Precast lightweight concrete panels**

Calsi-Crete, fireproof, lightweight cellular concrete is now available in panels for use as roof decks, walls, subfloors, etc.

Manufacturer states that planks are rugged, combining strength, stability, and insulation; are light enough for two men to handle easily and can be sawed, grouted, mortised or nailed. They are said to have a *K* factor of 0.81 and a *U* factor of 0.16.—Calsi-Crete, Inc. 6306 North Cicero Ave., Chicago 46, Ill.

**Jobsite concrete tester**

The Forney Model FT-20 jobsite testing machine for concrete and concrete products was recently introduced. Standardly equipped with a manually operated two

stage automatic pump and equipped for testing 6 x 12-in. cylinders in compression, a group of accessories are available for various operations such as conversion to electrical operation; testing concrete blocks; concrete pipe cores; concrete and clay drain tile; cement and concrete cubes; concrete beams and lintels, and compression and modulus of rupture of brick. Bulletin FT-20 available.—Forney's, Inc., Tester Division, Box 310, New Castle, Pa.

#### Joining waterstop sections

Waterseals, Inc., manufacturer of vinyl plastic waterstops, is currently marketing the "Sealiron" for joining waterstop sections. The equipment, which operates on 110 volt AC, includes a simple rheostat for adjusting the iron to the temperature required for smooth sealing. Light in weight, manufacturer states it is equally efficient in sealing either "labyrinth" or "striptype" waterstops.—Waterseals, Inc., 6 South Clinton, Chicago, Ill.

#### Hydraulically controlled cutting head

Masonry saw with a cutting head that is raised and lowered hydraulically is being marketed by Champion Mfg. Co. Developer of the saw refers to this as the "Hydra-Matic" saw and states that it was designed in response to contractors' demands for a saw with a movable cutting head that does not require cranking, pulling and inserting of pins, and other such operations.

The unit, available with 14 or 20-in. blade guard, comes equipped with a 110/220 voltage selector switch; diamond lock for jam cutting; either 1½ or 2 hp, air cooled motor; and built-in protection against blade shattering.—Champion Mfg. Co., 2028 Washington Ave., St. Louis 3, Mo.

#### Portable drilling tool

Roto-Imp, a drilling tool which combines rotary action with rapid "jack-hammer" blows is being offered by Bill Jack Scientific Instrument Co. The pounding action of the tool operates at 4000 to 7000 impulses a minute while the drill bit revolves at 650 to 1200 rpm.

Tests of the tool made by the manufacturer indicate that an average rate of 1 in. per min is obtained in high-test 6000 psi prestressed concrete of maximum age. Tool is said to cut through reinforcing bars and high-carbon twisted wire mesh imbedded in concrete.

Drills holes from ½ to 4 in. in diameter; weighs 30 lb, and operates on 115 volt household current or from small portable 1.5 kilowatt gas generator, either AC or DC current.—Bill Jack Scientific Instrument Co., Solano Beach, Calif.

## Literature Available

**Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.**

THE CEMENT INDUSTRY IN EUROPE—30-page booklet prepared by the Organization for European Economic Cooperation setting forth 1957 statistics on the cement industry in Europe. Covers trends in supply and demand, production, employment, imports, and exports. In French and English, $1 per copy.—Francis M. L. Black, administrator, OEEC Mission, Suite 1223, 1346 Connecticut Ave., N. W., Washington 6, D. C.

PRESTRESSED CONCRETE NEWS—Published quarterly for distribution by Freyssinet affiliated prestressed concrete product plants, each issue features a prestressed structure erected in this country, describing pertinent details and construction procedure; also describes a major project constructed abroad, generally of the same type as the feature article.—Freyssinet Co., Inc., 62 William St., New York 5, N. Y.

BUILDING MATERIALS LIST—Issued January, 1959, this 156-page list contains summaries of reports on applicances and materials which have been examined primarily with reference to fire preventive and fire protective capabilities. Retardant fire classifications are given for column protection, floor or roof and ceiling construction, and walls and partitions.—Underwriters' Laboratories, 207 East Ohio St., Chicago 11, Ill.

RACKMAN UNITIZED BLOCK HANDLING SYSTEMS—Illustrated brochure describes how the Rackman unitized block handling systems by GoCorp work.—GoCorp, 432 Grace St., Adrian, Mich.

METHOD OF ESTIMATING VIBRATING SCREEN AREA REQUIREMENTS—Using deck location, material shape, and material weight factors, and screen dimensions in the basic mathematical formula, method of estimating vibrating screen area requirements is described in seven steps, supplemented with appropriate charts.—Overstrom & Sons, Inc., 2213 W. Mission Rd., Alhambra, Calif.

ELASTIZELL LIGHTWEIGHT CONCRETE (Technical Report 2326-24-T)—Technical report prepared by the University of Michigan for Elastizell Corp. and Inland Steel Products Co., joint sponsors of the investigation, to determine whether Elastizell was capable of carrying the same allowable live load as ordinary concrete when placed over Milcor Cellufor

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DIAMOND TOOL ASSOCIATES—Brochures and prices on complete line of concrete cutting blades.—Diamond Tool Associates, P. O. Box 85, 940 E. El Segundo Blvd., Hawthorne, Calif.

MEARLCRETE FORM CONCRETE (Technical Bulletin R411)—Bulletin describes Mearlcrete foam concrete for roof decks and roof insulation. Illustrates with photographs and charts, Mearlcrete and how it is used. Typical installations are also shown.—Mearl Chemical Corp., 220 Westfield Ave., W., Roselle Park, N. J.

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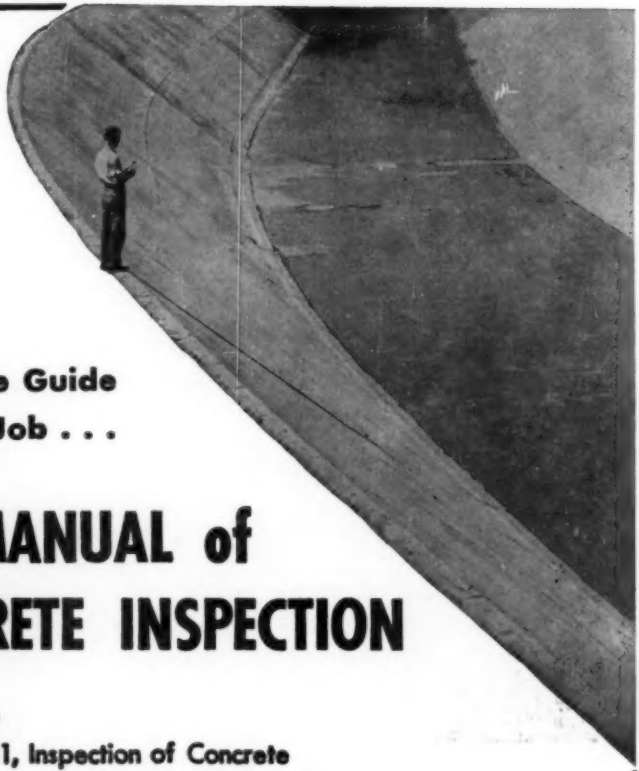

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